PRESTRESSED CONCRETE INSTITUTE

Vol. 4, June 1959 No. 1

JOURNAL

OF THE PRESTRESSED CONCRETE INSTITUTE

IN THIS ISSUE

	Page
EDITORIAL	17
SOME IMPORTANT PRESTRESSED SILOS AND RESERVOIRS IN SCANDINAVIA Chr. Ostenfeld & E. Kalhauge	18
PARTIAL PRESTRESSING AND POSSIBILITIES FOR ITS PRACTICAL APPLICATION Dr. P. W. Abeles	35
READER'S COMMENT	52
EFFECT OF TEMPERATURE DROP ON STRAND STRESSES IN A CASTING BED H. Kent Preston	54
TORSION IN CONCRETE PILES DURING DRIVING Ben C. Gerwick, Jr.	58
TESTS ON 15-YEAR-OLD, 54-FT. LONG, PRETENSIONED BEAM	64
BUILDING CODE REQUIREMENTS FOR PRESTRESSED CONCRETE	72

designed with imagination . . .
fabricated for trouble-free casting

watco

New forming concept is field tested in full size sections under actual yard conditions to determine pouring qualities and stripping capabilities. All WATCO forms are subjected to this critical field testing before going into production.

WATCO forms are available for all standard prestressed concrete structural sections. Engineering service offered to assist you in designing special forms for custom jobs.

Newest interpretation of one of the original prestressed concrete structural shapes is the Single (Lin) Tee. WATCO steel forms are now available to produce this simple, economical, easy-to-handle shape that has tremendous potential in the 60-125 ft. span range for floor and roof decking and bridge construction. Forms are designed for easy removal of cast sections with no cracking or stripping difficulties . . . no time-consuming form adjusting. For specifications and blue prints of this versatile new form . . . write

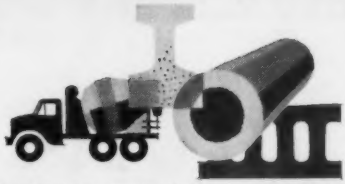
Manufacturers of WATCO Steel Forms

PLANT CITY STEEL CORP. • P. O. BOX 1308 • PLANT CITY, FLORIDA

CONVERTO MANUFACTURING COMPANY • CAMBRIDGE CITY, INDIANA

The number one choice for the pre-stressed industry!

TA680
P83



THE SMITH TURBINE-TYPE MIXER

The Smith Turbine Mixer—proved successful in over 100 installations—is a compact, lightweight, vibrationless mixer that can be easily installed in your batching operation.

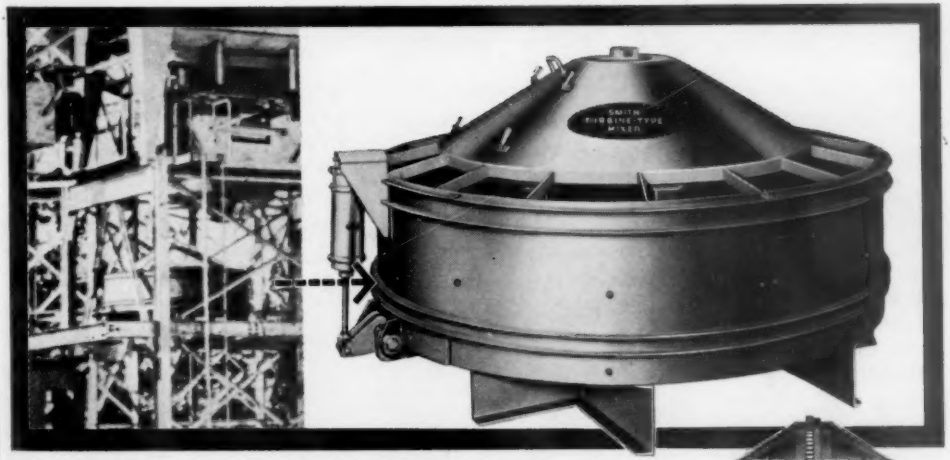
If you're building a new plant, use of the Smith Turbine permits lighter, lower, less expensive structures.

If you're converting an old plant, the Turbine can frequently be installed without extensive, major modifications — in places where no other mixer will fit!

Cycle time with The Smith Turbine is fast enough to charge up to 20 trucks an hour. What's more, the Turbine can discharge in four different directions, allowing you to alternate wet and dry mixes — handle pre-stressed ready-mix, block, or pipe batches all in the same machine.

Investigate the Smith Turbine. Write or call — we'll gladly tell you in detail how others are finding it the ideal way to beat competition while supplying higher-quality concrete.

Making dollars and sense in over 100 successful installations!



Since 1900, the pioneer designer and foremost manufacturer of the world's finest mixers

THE T. L. SMITH COMPANY

Milwaukee 1, Wisconsin • Lufkin, Texas

Affiliated with Essick Manufacturing Company, Los Angeles, Calif.

AS-4873-A

DICKERSON USES LESCHEN

Leschen Strand selected for one of the largest strand patterns tensioned in Pennsylvania

Dickerson Structural Concrete Corporation, Youngwood, Pennsylvania, is producing prestressed concrete box beams for highway bridges in Western Pennsylvania. The Allegheny County bridge shown utilizes beams 88'8" long by 4' wide by 3½' deep which weigh 37 tons each. Design plans called for seventy strands of ⅜" diameter in a pattern across the base and partially up the sides.

Dickerson is one of an increasing group of *Leschen Prestress Strand* customers who value the sound advice and practicality of Leschen engineers.

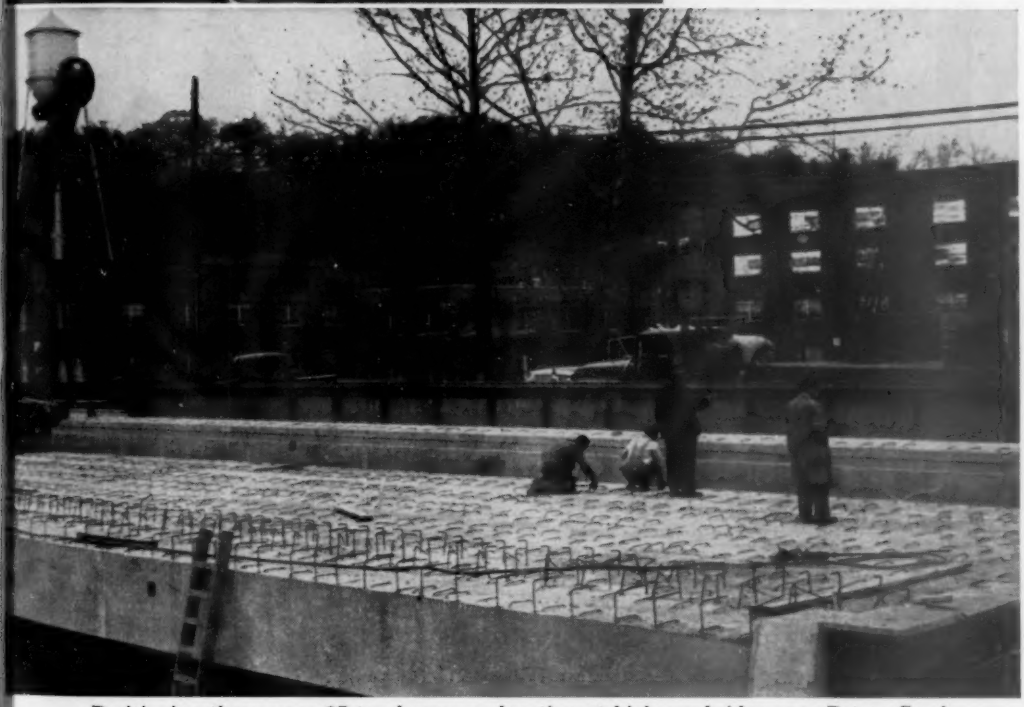
The technical requirements of prestressing, too, require unvarying strand quality—where every foot matches every other foot. At Leschen, this high standard is made possible by a new continuous-flow technique in today's most thoroughly modern wire mill. Here new processes, new machines, new methods maintain standards and quality that are above the industry's specifications.

If you would like to talk with a Leschen Strand engineer or receive complete specifications and other helpful information, write today to *Leschen Wire Rope Division, H. K. Porter Company, Inc., St. Louis 12, Mo.*

LESCHEN
PRESTRESS STRAND

Mr. Earle A. Butts, Jr., Dickerson Manager, looks over the strand template used in tensioning the Allegheny County strand patterns. He reports that Leschen Prestress Strand has performed as a well engineered and high quality product.

Leschen strand has helped the Dickerson plant to meet varying design requirements with box beams. The Allegheny County bridge beams were cast four at a time on a 440-foot bed, requiring over 30,000 feet of Leschen 3/8" Strand per pour. Tension: 14,000 pounds per strand or a total force of 980,000 pounds which is converted to a compressive force in the beam.



Positioning these same 37 ton beams on location at highway bridge over Peters Creek on T.R. 51 in Allegheny County near Pittsburgh. These beams with a minimum of construction depth, replaced an existing steel bridge.

LESCHEN WIRE PORTER ROPE DIVISION

H.K. PORTER COMPANY, INC.

PORTER SERVES INDUSTRY: with Rubber and Friction Products—THERMOID DIVISION; Electrical Equipment—DELTA-STAR ELECTRIC DIVISION, NATIONAL ELECTRIC DIVISION; Copper and Alloys—RIVERSIDE-ALLOY METAL DIVISION; Refractories—REFRACTORIES DIVISION; Electric Furnace Steel—CONNORS STEEL DIVISION, VULCAN-KIDD STEEL DIVISION; Fabricated Products—DISSTON DIVISION, FORGE AND FITTINGS DIVISION, LESCHEN WIRE ROPE DIVISION, MOULDINGS DIVISION; and in Canada, Refractories, "Disston" Tools, "Federal" Wires and Cables, "Nepcoduct" Systems—H. K. PORTER COMPANY (CANADA) LTD.

when the job is done
and all the facts are in—
you'll find
STRESSTEEL
overall cost is lowest!

110' bridge girders, 4'2" high and 2' wide with 8" web. Cherry Avenue Overhead, vicinity Los Angeles, Cal. Precast in yard of Concrete Conduit Company, Colton, Cal., subsidiary of American-Marietta Co. E. L. Yeager, general contractor for Division of Highways, Department of Public Works, State of California.

There is reason for the tremendous growth in the use of **STRESSTEEL** for prestressed concrete. Increasing numbers of users find . . .

- **STRESSTEEL** is the lowest in-place cost post-tensioning material.
- **STRESSTEEL** handles faster, providing lower labor costs for placing and tensioning.
- **STRESSTEEL** proof-tests every bar, assuring controlled quality.
- **STRESSTEEL** Wedge Anchor Units provide positive end anchorage. It is the fastest, safest and easiest post-tensioning method.
- **STRESSTEEL** tensioning equipment for rent and sale at nominal cost.

Write for new illustrated manual SS-3

STRESSTEEL CORPORATION
221 Conyngham Ave.
WILKES-BARRE, PENNA.

Use Richmond Inserts in Prestressed Concrete

Placing of Amdek hollow box girders by means of *Richmond* Lifting Inserts with loads spread and equalized.

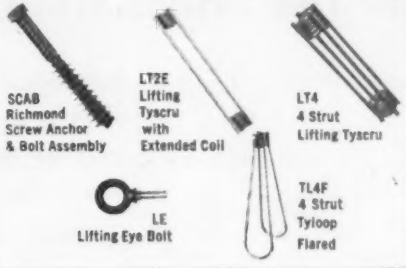
Versatility and Strength of Richmond Lifting, Anchoring and Fastening Inserts Assures Economy and Safety

The ever increasing use of prestressed concrete has created a great need for a variety of special inserts for lifting, anchoring and fastening to prestressed concrete. Richmond has developed and tested a complete line of Lifting Inserts and Structural Concrete Inserts for prestressed columns, beams, girders, piles, etc. These inserts are designed with ample strength and for simple operation. Certified laboratory tests assure proper strength rating for efficient design and performance in prestressed concrete at usable strength.

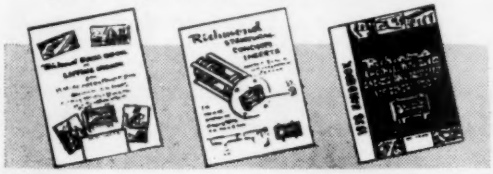
Available in ½" to 1½" diam. in strengths to 65,000 lbs. in 3,000 psi concrete

New Richmond Structural Concrete Inserts

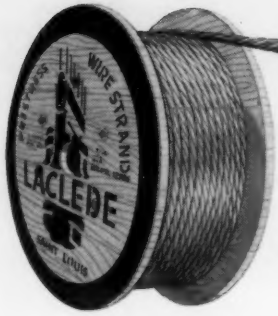
a recent addition, these inserts are prefabricated from a special design which distributes the bolt stresses into the concrete for greater strength than any previously known device.



The Richmond Data Book on Lifting Inserts and the new Richmond Bulletin on Structural Concrete Inserts give complete information about these and the many other Richmond Inserts specifically designed for safety and economy. Send for them — and for the current Richmond Handbook describing the full line of more than 400 Richmond-engineered products for the concrete construction industry. They are free for the asking. Write to:



INSIST ON RICHMOND
....AND BE SURE
IT'S RICHMOND
TIME & LABOR SAVING CONSTRUCTION DEVICES FOR CONCRETE CONSTRUCTION
Richmond
SCREW ANCHOR CO., INC.
816-838 LIBERTY AVE. BROOKLYN 8, N.Y.
315 SOUTH FOURTH ST. ST. JOSEPH, MO.



LACLEDE OPEN HEARTHS

. . . are the starting point for the finest steel for prestressing

For years, Laclede has been recognized for the quality of its products made possible by the experience and know-how of the Laclede Metallurgists and Operating men in the making of the steel.

Laclede 7 Wire Strand for Prestressing is a typical example of this product quality beginning at the open hearth, that assures the prestresser that the most exacting design requirements will be satisfied by the finished strand.

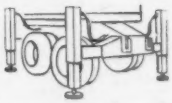
LACLEDE

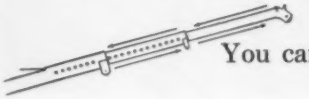
It's the STEEL that puts the Quality in the Strand

LACLEDE STEEL COMPANY

GENERAL OFFICES: *St. Louis, Missouri*
DISTRICT OFFICES: *Chicago, Illinois* · *Dallas, Texas* · *Detroit, Michigan* · *Houston, Texas* · *Kansas City, Missouri* · *Memphis, Tennessee* · *Moline, Illinois* · *New Orleans, Louisiana* · *Tampa, Florida*

This is a HYDROCRANE!

It will lift 24,000 pounds. Its outriggers  operate hydraulically and individually — within seconds to keep you level when you work. The controls are hydraulic — oil-smooth. The boom on a Hydrocrane is telescoping.

 You can add 12 feet to your reach *on the move*, without stopping! The standard boom can be 24 to 36 feet long, the high lift boom can be 38 to 50 feet long — or even 70 feet long with the jib! The Hydrocrane travels over streets, highways, in plant yards as easily and fast as your pickup trucks. Does it work? It works so well that over 50% of the customers* who have ever bought a Hydrocrane, have bought another!

*Who are they? They're precast concrete people, municipalities, utility companies. They're contractors, loggers and steel erectors. And, there are many more every day. See your **Hydrocrane dealer for a free demonstration,** or write Dept. 18H, Bucyrus-Erie Company, South Milwaukee, Wisconsin.

BUCYRUS ERIE®

Builds Better Equipment

QUALITY BRANDED, FULLY GUARANTEED PRESTRESS WIRE AND STRAND

Who makes it? Sumitomo Electric Industries, Ltd., the largest manufacturer of wire and cable in the far east — established in 1897 — and known the world over.

What about quality? All wire and strand is made to ASTM specifications and fully guaranteed—your assurance of high quality.

Who imports & distributes it? We do — Kurt Orban Company, Inc. We've imported a million tons of steel and steel products in the last ten years, plus all types of precision machine tools and industrial equipment. We've supplied some 2000 accounts — including some of the best known names in this hemisphere—with everything from nails to complete plant installations.

Who guarantees it? We do — Kurt Orban Company, Inc. Our reputation, our experience and our resources are behind this guarantee. A Kurt Orban representative is always just a phone call away.

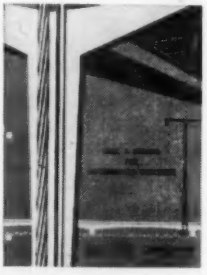
Who buys it? Major prestress contractors in the U.S. have ordered and re-ordered, contractors who value quality. How about you?

KOC products for Prestressors:

Wire—ASTM A-421-58T
Strand—ASTM A-416-57T
Test Certificates Supplied
Wire rods—ASTM A-15-54T
Bright wire—ASTM A-82
Reinforcing bars—ASTM A-15-54T and ASTM 305

Send for this booklet today

Plus "How to Be At Home With Products Made Abroad", the story of the Kurt Orban Company, Inc.



PRESTRESSED CONCRETE INSTITUTE

KO KURT ORBAN COMPANY, INC.

Member of Prestressed Concrete Institute

34 Exchange Place, Jersey City 2, New Jersey

Voided bridge piles save money!

Tampa Bay Bridge, Florida
Hardaway Contracting Company, contractors
State of Florida Bridge Dept., engineers and designers.

Precast, prestressed concrete piles contain

SONOCO SONOVOID FIBRE TUBES

The Tampa Bay Bridge connects Tampa and St. Petersburg, Florida and is a 3-mile long structure accommodating four lanes of traffic.

Piles for the project are 24" square, precast prestressed concrete containing 12" O.D. Sonoco SONOVOID Fibre Tubes and twenty-four 7/16" seven-strand pretensioning cables.

There are 285,000 linear feet of piles varying in length from 60 to 110 feet. The average pile length is 75 ft. and despite these rather long piles, it is reported that the cost of the structure is very low.

Sonoco SONOVOID Fibre Tubes were specifically developed for use in bridge decks, floor, roof and lift slabs and in concrete piles. For prestressed precast units or units cast in place.

Order in sizes from 2.25" to 36.9" O.D. up to 48' long.
Specify lengths to meet your needs or saw to size on the job. End closures available.

See our catalog in Sweet's

For complete information and prices write

- HARTSVILLE, S. C.
- LA PUENTE, CALIF.
- MONTCLAIR, N. J.
- AKRON, INDIANA
- LONGVIEW, TEXAS
- ATLANTA, GA.
- BRANTFORD, ONT.
- MEXICO, D.F.

SONOCO
Construction Products
SONOCO PRODUCTS COMPANY

3233-A

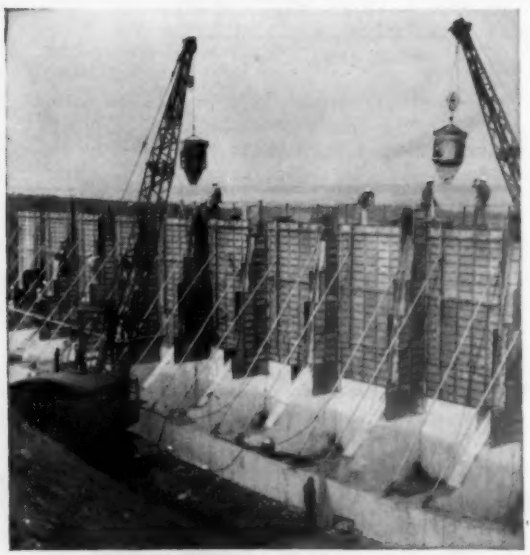
CONCRETE PERFORMANCE REPORT:

POZZOLITH concrete employed in new Air Force Academy to meet full range of engineering requirements for all types of concrete specified

Largest single construction project in U. S. Air Force history, the U. S. Air Force Academy at Colorado Springs marks a milestone in modern concrete design and construction. The project included the placing of some 800,000 cubic yards of concrete for buildings, retaining walls and bridges.

On-site concrete control lab—The Air Force Academy Construction Agency and the architects—Skidmore, Owings and Merrill—jointly supervised all construction and established an on-site concrete materials control laboratory early in 1956. During July and August, 1956—with only a few

AIR FORCE



CONCRETE RETAINING WALLS reach 36 feet high over much of the 10,000 foot wall length. Tallest pours were made by giant overhead crane. Walls required approximately 24,000 cubic yards of POZZOLITH concrete—a placeable mix of 2″ to 4″ slump with design strength of 3,000 psi. Contractor: T. F. Scholes, Inc., Reading, Pennsylvania. Concrete Contractor: Long Construction Co., Denver.

thousand yards of concrete placed—they observed erratic and low compressive concrete strengths. The wide range and rapid changes in temperature were suspected as the cause.

Evaluation tests of concrete materials—In August, they engaged Commercial Testing Laboratories, Denver, to make comprehensive tests of the concrete materials under the range of temperatures being experienced at the job site. Their tests clearly established that POZZOLITH would provide uniform, high strength throughout the wide range of temperature changes experienced between early morning concreting at about 50°F and midday concreting at 75° to 80°F. In September 1956, POZZOLITH was first employed in concrete at the Academy. The successful performance of POZZOLITH here led engineers to investigate the use of POZZOLITH for control of other classes of concrete—including lightweight aggregate concrete, prestressed concrete and structural concrete. *As a result of this investigation,* POZZOLITH *and only* POZZOLITH *was used as the water-reducing, set-controlling admixture for the project.*

POZZOLITH and Master Builders field service—POZZOLITH was an important aid

AERIAL VIEW of nearly completed Air Force Academy. Construction under supervision of the Air Force Academy Construction Agency. Architects: Skidmore, Owings & Merrill, Chicago • Over 24 contractors were engaged in major concrete work. Ready-mixed concrete was supplied from central batch plants operated by: Concrete Materials, Inc., Kansas City • General Concrete Co., Colorado Springs • Transit Mix Concrete Co., Colorado Springs.

ACADEMY

in meeting and exceeding specification requirements in over 750,000 of the 800,000 cubic yards of concrete at the Air Force Academy. For each type of concrete specified — it provided the required batch-to-batch uniformity, most economically, for the broad range of job requirements and varied climatic conditions encountered at the site.

The Master Builders field men and the Company engineering staff worked closely with project engineers, the field control laboratory, contractors, and concrete suppliers to achieve the common goal of uniform, superior quality concrete at lowest cost-in-place.

Neither plain concrete nor concrete with any other admixture can match the results you obtain with today's POZZOLITH. The local Master Builders field man will welcome discussing your requirements. Call him in. He's at your service—backed by the Master Builders research and engineering staff—unexcelled in the field of concrete technology. Write us for complete information on POZZOLITH.

And write for your free copy of the *detailed*
"Air Force Academy Concrete Performance Report".

The Master Builders Company, Cleveland 3, Ohio • Division of American-Marietta Company
The Master Builders Company, Ltd., Toronto 15, Ont. • International Department, New York 17, N.Y.
Branch Offices in all principal cities.

MASTER BUILDERS®
POZZOLITH*

***POZZOLITH** *is a registered trademark of The Master Builders Company for its concrete admixture to reduce water and control entrainment of air and rate of hardening.*

FOR POST TENSIONING . . . *Specify*

DUOFLEX

CASING

DUOFLEX casing is preferred by leading manufacturers of tensioning elements for encasing post-tensioning bars, strand or cable. Field tested throughout the world by prestressing contractors, DUOFLEX offers economy in first cost and in trouble-free handling on the job.

DUOFLEX casing is rugged; resists crushing, and its specially designed contour creates a lasting, mechanical bond with the concrete. For best results in post-tensioning, get the patented DUOFLEX casing, and keep your bids low. Write for specifications and price list.

Patent No.
2,832,375

PRESTRESSED CONCRETE INSTITUTE

FLEXICO PRODUCTS, INC.

Flexible Metal Hose and Tubing for Industry

Phone LIBERTY 8-5100
P. O. Box 484
METUCHEN, NEW JERSEY

SHAKE, RATTLE and RÓLL

any Bin, Chute, Weigh Batcher, with a . . .

TOPDOG
electric vibrator
EVEN USE AS A CAR SHAKER

- **A size for any bin**
- **Outside operation, all year 'round**
- **Totally enclosed, non-vented**
- **Low maintenance — grease every 1,000 operating hours**
- **6 months guarantee**

Topdog electric vibrators meet the needs of the Ready-Mix industry. They can be set-up to operate on most any type of current. Write for full details and literature.

VIBRO-PLUS
PRODUCTS, INC.
STANHOPE, NEW JERSEY

WORLD'S LEADING MANUFACTURER OF VIBRATORY EQUIPMENT FOR OVER TWO DECADES !

41-84

They're using

CF&I Prestressed Concrete Strand and Wire

The CF&I Image stands for lasting strength which is built into quality steel. On construction jobs this symbol represents all CF&I steel products. And two of them—CF&I Prestressed Concrete Strand and Wire—are especially important to makers of prestressed concrete members.

For pretensioning of concrete members—use CF&I Prestressed Concrete *Strand*. This 7-wire strand is stress relieved after stranding to insure the proper elastic properties. Made in accordance with ASTM A416-57T.

For post-tensioning of concrete members—use CF&I Prestressed Concrete *Wire*. A high grade wire with excellent elastic and uniform physical properties. Rigid quality controls result in a wire that lies flat and straight for easy handling and working . . . make it ideal for Button and Wedge type Anchorage methods of post-tensioning.

For complete information about any CF&I steel product, contact our nearby sales office.

Other CF&I Products for Concrete Construction

Clinton Welded Wire Fabric ASTM A185-56T • CF&I Concrete Reinforcing Bars and Rods ASTM A15-54T • CF&I Wire for Concrete Reinforcement ASTM A82-34 • CF&I MB Spring Wire ASTM A227-47 • Cal-Tie Wire • Wickwire Rope and Slings

6629-A

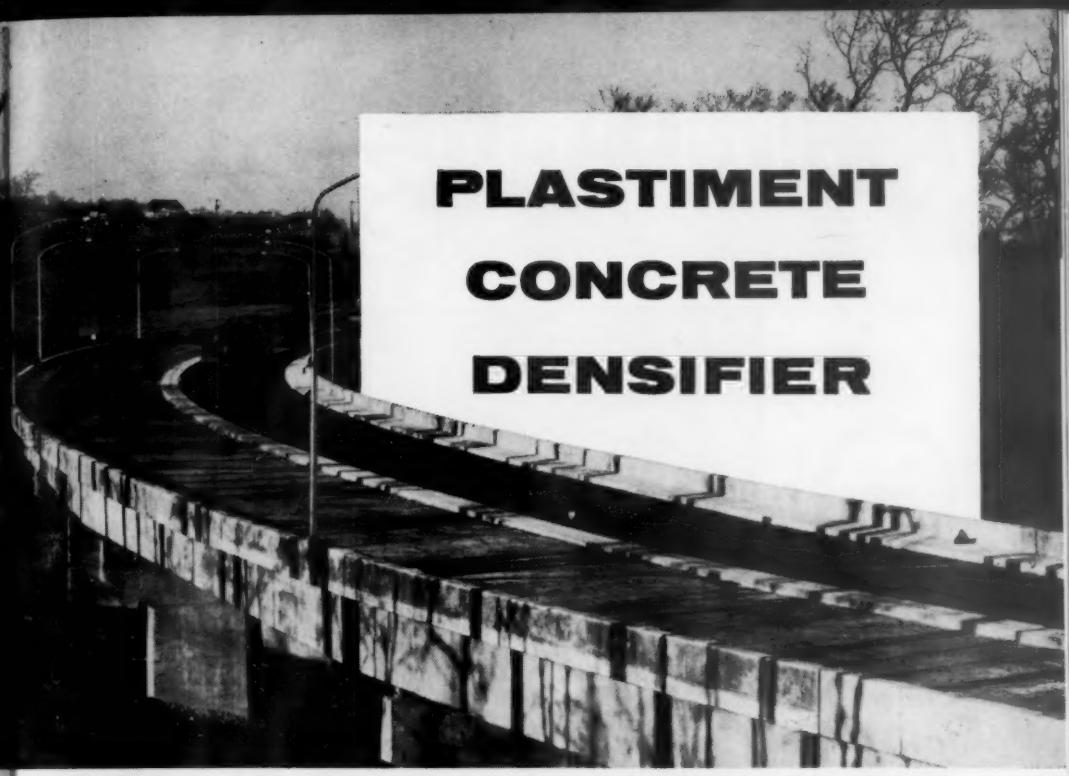
PRESTRESSED CONCRETE STRAND AND WIRE

THE COLORADO FUEL AND IRON CORPORATION

In the West: THE COLORADO FUEL AND IRON CORPORATION—Albuquerque • Amarillo • Billings Boise • Butte • Denver • El Paso • Farmington (N. M.) • Ft. Worth • Houston • Kansas City • Lincoln Los Angeles • Oakland • Oklahoma City • Phoenix • Portland • Pueblo • Salt Lake City • San Francisco San Leandro • Seattle • Spokane • Wichita

In the East: WICKWIRE SPENCER STEEL DIVISION—Atlanta • Boston • Buffalo • Chicago Detroit • New Orleans • New York • Philadelphia

CF&I OFFICE IN CANADA: Montreal CANADIAN REPRESENTATIVES AT: Calgary • Edmonton Vancouver • Winnipeg



PLASTIMENT CONCRETE DENSIFIER

Specified FOR PHILADELPHIA'S GORGAS LANE BRIDGE

Plastiment concrete in 7 foot deep beams is being revibrated before deck is placed.

This unusual monolithic prestressed beam and slab design resulted from alternate bids permitted by Philadelphia's progressive Department of Streets* . . . Savings of more than 20% over conventional steel design were passed on to the taxpayers.

Each 120 ft. long span, 34 ft. wide; including two traffic lanes, sidewalk, and half of the center median strip was concreted in one continuous 260-yard operation . . . Since 8-10 hours elapsed during placement, Plastiment was specified to control initial set. Plastiment provided these additional benefits: better compaction, greater density, and faster development of stressing strength.

Your clients will benefit from the better structural quality of Plastiment concrete . . . Specify Plastiment for your next job . . . Write for Bulletin PCD.

AD 26-7

*David M. Smallwood, Street Commissioner
Noel W. Willis, Chief Bridge Engineer

The Preload Company, Inc., New York City—Consulting Engineers
The Conduit & Foundation Corporation, Philadelphia—Contractor

SIKA CHEMICAL CORPORATION

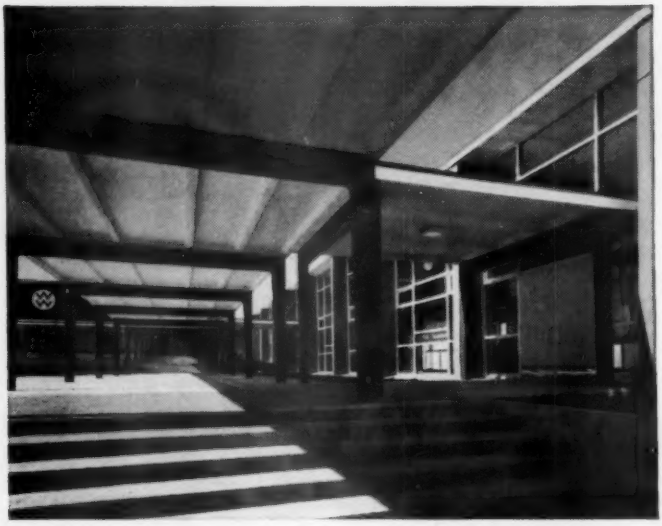
PASSAIC, NEW JERSEY

DISTRICT OFFICES: BOSTON • CHICAGO • DALLAS • DETROIT • PHILADELPHIA • PITTSBURGH • SALT LAKE CITY • WASHINGTON • DEALERS IN PRINCIPAL CITIES—AFFILIATES AROUND THE WORLD

Sika

TACOMA

Rings the Bell!



● "As close to perfection as one could desire for student evacuation and for reducing the spread of fire." That's how the head of Tacoma's Fire Prevention Bureau described this new, all-concrete Woodrow Wilson High School.

Detailed cost comparisons, including savings on insurance premium costs, led to concrete construction. Among the 491 prestressed concrete girders, made by the Concrete Technology Corporation, were 105-foot spans over the gymnasium—the largest prestressed beams in the Pacific Northwest. All columns, tilt-up wall panels, walkway frames, and some 6,000 channel roof slabs were precast at the job site. 'Incor' was used exclusively in all prestressed members, and Lone Star Portland in all other concrete work.

Yes, Tacoma has rung the bell with this prefabricated concrete school. For whom does the bell toll? School authorities across the nation, faced with inadequate, deteriorating, flammable structures—faced with public demands both for safer schools and greater economy—may well heed the ringing!

LONE STAR CEMENT CORPORATION

Offices: ABILENE, TEX. · ALBANY, N. Y. · BETHLEHEM, PA. · BIRMINGHAM · BOSTON · CHICAGO · DALLAS · HOUSTON · INDIANAPOLIS
KANSAS CITY, MO. · LAKE CHARLES, LA. · NEW ORLEANS · NEW YORK · NORFOLK · RICHMOND · SEATTLE · WASHINGTON, D. C.

LONE STAR CEMENT, WITH ITS SUBSIDIARIES, IS ONE OF THE WORLD'S LARGEST CEMENT PRODUCERS, 21 MODERN MILLS, 49,100,000 BARRELS ANNUAL CAPACITY

JOURNAL

OF PRESTRESSED CONCRETE INSTITUTE

Vol. 4 June 1959 No. 1

OFFICERS AND DIRECTORS
1958-1959

Peter J. Verna, Jr.
President

Philip E. Balcomb
Vice President

Jacob O. Whitlock
Secretary-Treasurer

DIRECTORS

Douglas P. Cone
Randall M. Dubois
George W. Ford
Ben C. Gerwick, Jr.
J. Ashton Gray
R. O. Kasten
Prof. T. Y. Lin
John K. Zimmerman

EXECUTIVE SECRETARY
Martin P. Korn, M.ASCE

CHAIRMAN TECHNICAL ACTIVITIES COMMITTEE
Charles C. Zollman

•

The INSTITUTE is not responsible for any statements made or opinions expressed in its publications.

EDITORIAL

This issue contains several original and provocative papers which might be valuable for reference material. "Prestressed Silos and Reservoirs in Scandinavia" is alive with illustrations of general interest and creative design. Dr. Abeles' paper "Partial Prestressing and Possibilities for its Practical Application" will prove instructive to many and controversial to some who do not believe in partial prestressing. The JOURNAL'S Readers' Comment section invites discussion.

An Active Member of PCI presents a problem on prestressed piling which probably has troubled others. It is published in the current Readers' Comment with several solutions. Of pertinent and timely interest is the paper by Ben C. Gerwick, Jr., an authority on prestressed piling.

The Structural Engineers Association of Northern California, a most competent and progressive group, have drawn up a proposed Building Code to be embodied in the well-known Uniform Building Code on the West Coast. We are indebted to them for permission to publish it. If these papers have given you a big lift, do not stop until you read "Effect of Temperature Drop on Strand Stresses in a Casting Bed" and "Tests on a 15-year-old, 54 ft. long pretensioned beam." You can then pause for fruitful meditation.—M.P.K.

The **JOURNAL** is published quarterly by the **PRESTRESSED CONCRETE INSTITUTE, INC.**, 425 N. E. Fifth Street, Boca Raton, Florida. **ADVERTISING, SUBSCRIPTION** and **PUBLICATION OFFICE:** 3132 N. E. Ninth Street, Fort Lauderdale, Florida. **DISTRIBUTION:** Available without cost to all members of the Prestressed Concrete Institute. **SUBSCRIPTION PRICES:** $6.00 per year for four issues. Single copy price: $2.00. For subscriptions outside the continental United States, add 20% to cover handling and mailing. Editorial contributions to the **JOURNAL** are welcomed, but publication cannot be guaranteed. All manuscripts should be submitted in duplicate and will be reviewed by the Technical Editor prior to publication. Advertisements of products, services and materials allied to the prestressed concrete industry are acceptable. Inquiries concerning advertising, and subscriptions should be directed to the Publication Office.

Copyright by the **PRESTRESSED CONCRETE INSTITUTE, INC.**, 1959.

some important prestressed silos and reservoirs in Scandinavia

BY

CHR. OSTENFELD, DR. TECHN., M.ASCE
E. KALHAUGE, CIVIL ENGINEER

Chr. Ostenfeld & W. Jonson,
Consulting Engineers,
Copenhagen, Denmark

INTRODUCTION

The construction of circular-cylindrical prestressed concrete tanks was started in U.S.A., in a primitive form, already before the last world war (Hewitt-tanks for water storage). The invention of prestressing by wire-winding (Preload) gave during the war a considerable raise to the construction of prestressed tanks for storage of oil and water. After the war this type of tank has maintained an important position in U.S.A., especially for water storage tanks, whereas the construction of oil storage tanks in prestressed concrete has been almost abandoned since steel again became cheaper and less scarce.

In Europe the application of prestressed concrete for tanks had to wait until the years after the war; the European development of such structures has been different from the American development in two respects:

a) Prestressed circular-cylindrical structures have in Europe been applied to the storage of many different materials. Thus tanks have been built for liquids such as water (in some cases boiling hot), various products of mineral oils, cement slurry etc., and silos have been built for granulated materials such as cement, sugar, coal etc. The following chapters of this article will give a brief description of some typical or outstanding specimens of the tanks and silos constructed in Scandinavia which is the European region where the activity in this field hitherto has been the most important.

b) The Scandinavian tanks and silos are normally designed as monolithic concrete structures, without sliding joint or similar disconnection between the wall and the bottom slab, and prestressed by means of cables or bars placed inside the concrete. Fig. 1 shows in a simplified form the main features of this type of tank (or silo).

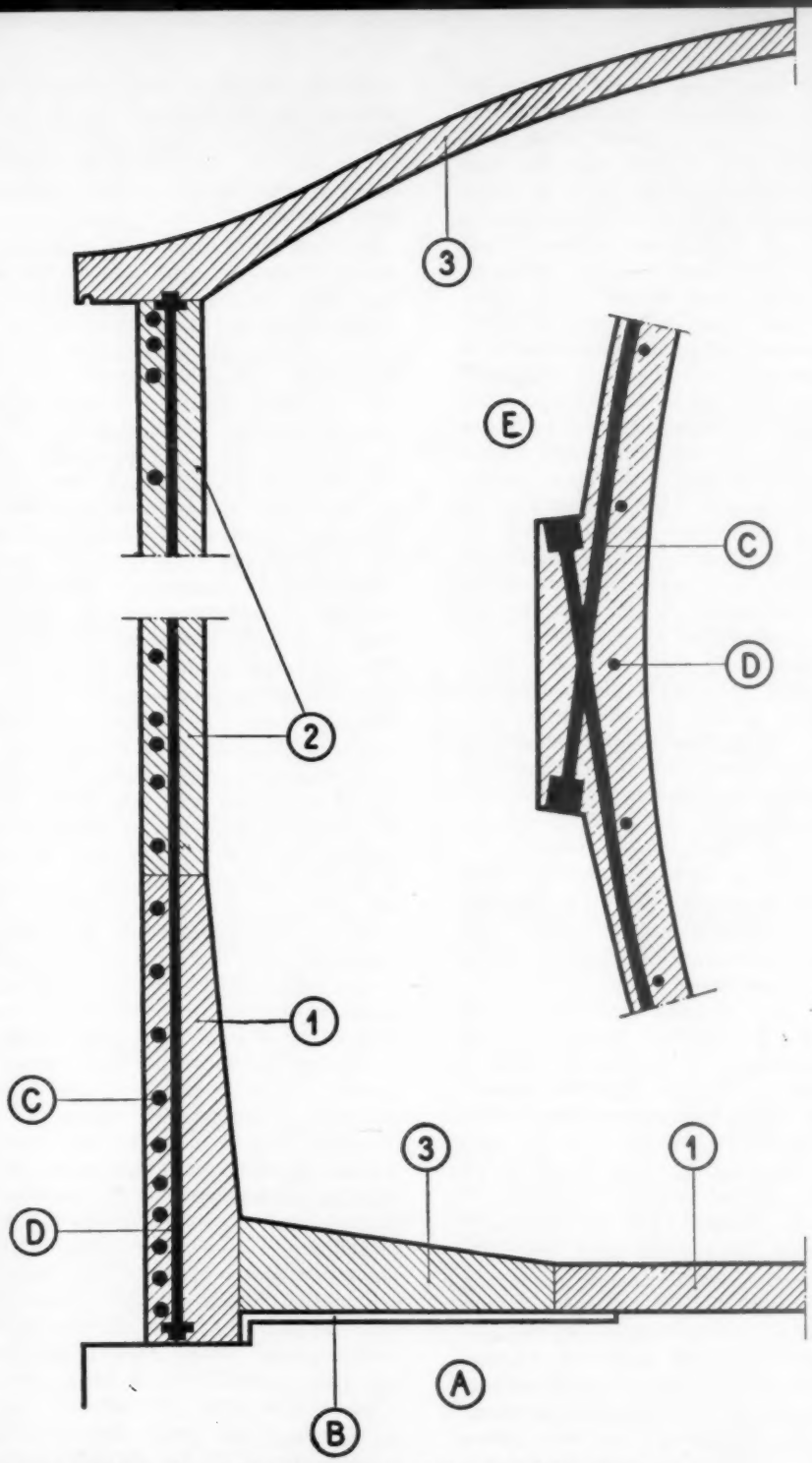


Figure 1. Sections of typical, big reservoir of the monolithic type.
1-3 Stages of pouring the concrete.
A. Foundation
B. Soft layer (only by rigid foundations)
C. Horizontal prestressing cables (Normally composed of 0.2-0.3″ wires)
D. Vertical prestressing cables (normally 1″ bars)
E. Horizontal section of wall at pilaster for anchoring of horizontal prestressing cables

There have been several reasons for choosing a prestressed reinforcement placed in concrete, in the same way as it is usual for beams, slabs, and the like. The most important reason was the desire to reduce maintenance costs to a minimum, even in the difficult Scandinavian climate, where frost and thaw may alternate frequently during rather long periods of the year. Under such conditions, almost any layer of mortar, protecting the reinforcement of a wire-wound reservoir will be susceptible to peeling after a while. This is better understood when it is realized that the protective mortar will be subject to tensile stresses which are the sum of the stress due to filling the reservoir (these stresses may attain 1000 psi by big reservoirs) and of the initial tensile stress in the mortar due to the fact that the greater part of the shrinkage of the mortar is precluded by the much thicker wall of older concrete.

Opposite hereto there is in the monolithical wall no inherent plane separating concrete of different ages to provoke peeling, and moreover the residual compressive stress exists all through the concrete section even when the reservoir is filled. Finally the prestressed reinforcement can be placed fairly deep in the concrete (2" clearance to the surface is normal), which is of great importance when the fire-resistance of e.g. oiltanks is regarded.

Another advantage obtained by using cables for prestressing is that a slight compressive stress — sufficient to prevent the occurrence of shrinkage cracks — could be introduced in the structure, in whatever part desired, a couple of days after pouring the concrete (until then a good water-curing will postpone the shrinkage). In this connection it should be mentioned that big reservoirs are fairly exposed to harmful effects of the shrinkage before prestressing, as they at this time actually are unreinforced.

The possibility of introducing prestressing with any desired force at any place in the structure, and at any time during the construction, facilitates the realization of a final monolithic connection between wall and bottom slab (see fig. 1). Pouring and prestressing in stages is a powerful means to govern and reduce the moments which occur in the neighbourhood of this connection. Another means in this respect is to create, artificially, conditions of the support which attenuate the said moments, e.g. by placing a soft layer between the foundation and the bottom slab (B on fig. 1); this method is especially used for big reservoirs having a rigid foundation.

After all the difficulties of producing a monolithic connection between the wall and the bottom slab are the headache of the consultant and not of the contractor. If the consultant takes the care to solve this problem (which only can be done in a successful way by taking all statical conditions into account, including variating thickness, construction in stages, elastic behaviour of the foundation etc.) the client will obviously get the best possible construction. Even the best joint between wall and bottom (whether sliding or fixed or made tight with special sealers) must be considered as inferior to a monolithic construction, and such a joint is likely to demand maintenance in time.

A few words should be said about the friction losses in prestressing force of the horizontal cables. Fig. 2 shows schematically (as a supplement to fig. 1, E) an evolution of a wall with 4 vertical pilasters for anchoring of the cables, and the corresponding arrangement of the cables (in reservoirs with big diameter 6 and 8 pilasters may be used). It will be seen that when successive cables are anchored on alternating pilasters, the resulting average prestressing force will be practically linear. In this connection it should be mentioned that the friction in the horizontal cables will normally not increase the amount of prestressing steel necessary to take up the horizontal thrust from the material in the reservoir without tensile stresses in the concrete. The amount of steel is a priori fixed by the safety factor against rupture, which normally is required to be about 2, in accordance with other reinforced or prestressed structures. Therefore, it will be sufficient to produce an effective mean prestressing that is slightly higher than the half of the ultimate strength of the steel, in order to avoid tensile stresses in the concrete under normal working conditions, and such a prestressing is fairly easy to obtain in spite of the friction.

Concluding this introduction the construction stages of the typical reservoir (tank or silo) on fig. 1 should in short be mentioned and commented upon:

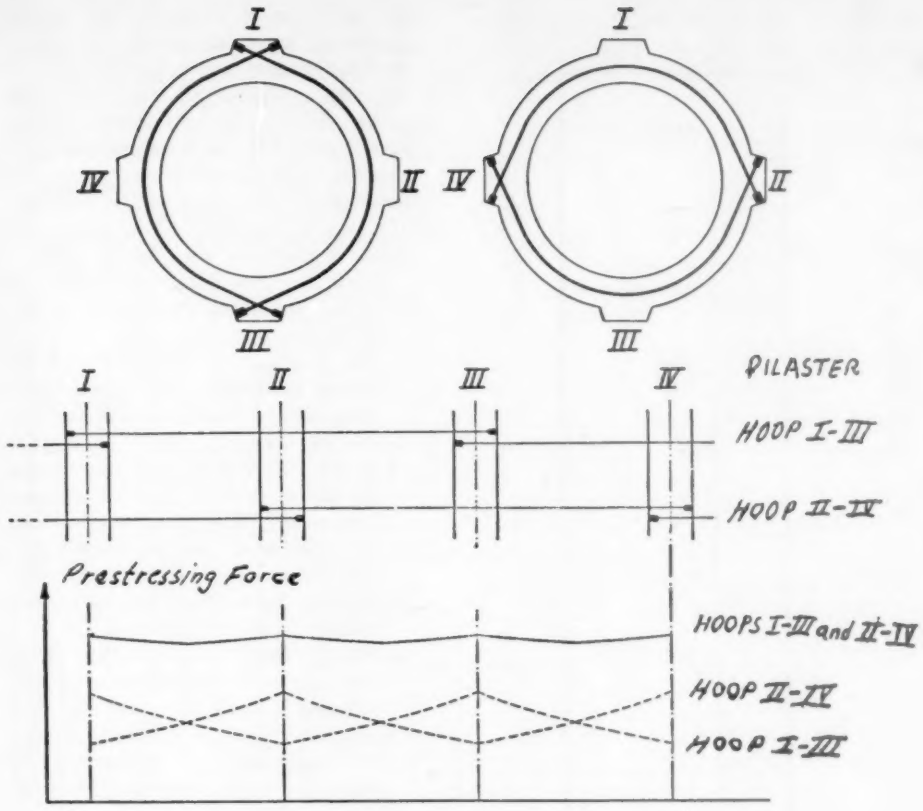


Figure 2. Schematic representation of horizontal prestressing by cables anchored successively in alternating pilasters. Evolution of the wall of the reservoir, showing how the resulting ring force from the prestressing will be nearly uniform.

a) The basic part (1) of the wall, with a height 3' to 5' is poured continuously in conventional formwork, (in big reservoirs this part has a decreasing thickness upwards), and is given a slight horizontal prestressing after a couple of days, in order to avoid shrinkage cracks. The central part of the bottom (1) may be poured at about the same time using a fairly dry concrete which .is submitted to surface vibration in several hours whereby is obtained a very strong concrete, almost without shrinkage. The bottom is water-cured under a thin layer of water.

b) The current part (2) of the wall is poured continuously with the use of slip forms whenever the height exceeds about 20'. As soon as the wall has attained a height of about 10' part 1 is further prestressed, again with the aim to counteract the shrinkage. The problems related to placing the horizontal cables in the slip forms were at an early date solved in a simple and satisfying way, and here should only be mentioned that flexible cables composed of wires (3/16"-5/16"), are the most practical for this job.

If the size of the reservoir makes a verti-

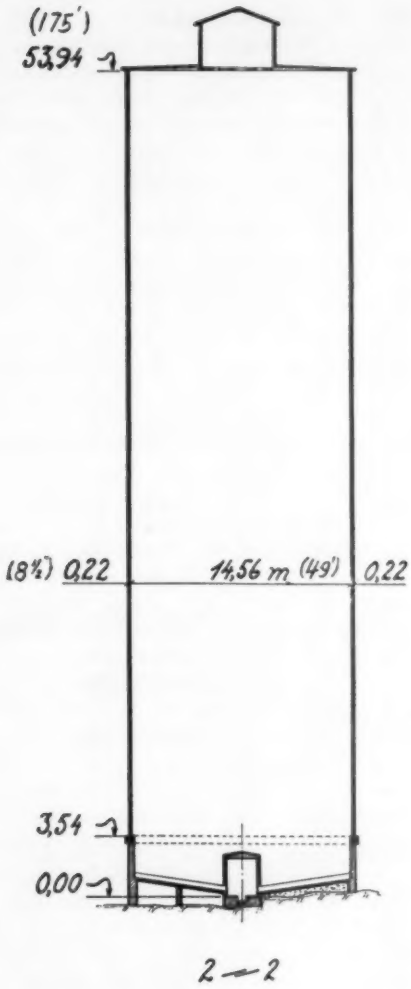


Figure 3. 175' cement silo in Norway.

cal prestressing desirable as a supplement to the mild reinforcement (nearly all silos and most part of the smaller tanks are designed without vertical prestressing) bars with threaded ends anchored with nuts and anchorplates, and if necessary prolonged by means of special couplings will be the most convenient for this purpose. This is especially the case when slip forms are used because the bars can be erected in short lengths (10-15') during the lifting.

c) The prestressing of the wall, vertical, if any, and horizontal, is completed with the exception of a certain part of the horizontal prestressing near the base. This last prestressing is first carried out after the remaining annular part of the bottom slab (3) has been poured continuously, and this prestressing will thus introduce compressive stresses in the construction joints between bottom and wall. The bottom slab will at the same time obtain a moderate compressive stress.

It will be seen that the advantageous effects of postponing the pouring of part 3 are triple:

i) The shrinkage of the wall and the bottom may take place without creating harmful stresses as wall and bottom are not connected.

ii) As the wall is free at its base during the construction it can be given a compressive ring stress at the base also, this would only be possible to a very small extent if the connection between the wall and the bottom was created from the beginning.

iii) The fact that the prestressing of the wall can be applied to two statically different systems (free wall and wall connected with the bottom) increases the possibilities of attenuating the moments at the connection between wall and bottom.

The above-mentioned general considerations will in the following chapters be illustrated by a brief survey of some typical or outstanding tanks and silos constructed in Scandinavia. The division of the description in groups follows an industrial line.

CEMENT INDUSTRY

Fig. 3 and 4 show a cement silo constructed in the northern part of Norway, at the Nordland Portland Cement factory (located at a desolate fjord above the polar circle). With a total height of 175', a diameter of 49' and holding nearly 15,000 tons of cement, it is probably the biggest cement silo hitherto constructed in prestressed concrete.

The prestressed wall, 8½" thick, was poured in slip forms which climbed 165' in 3 weeks. The slip forms were combined with the self-supporting scaffolding for the roof construction, thus creating a spacious working platform during the sliding, and permitting the pouring of the roof shortly

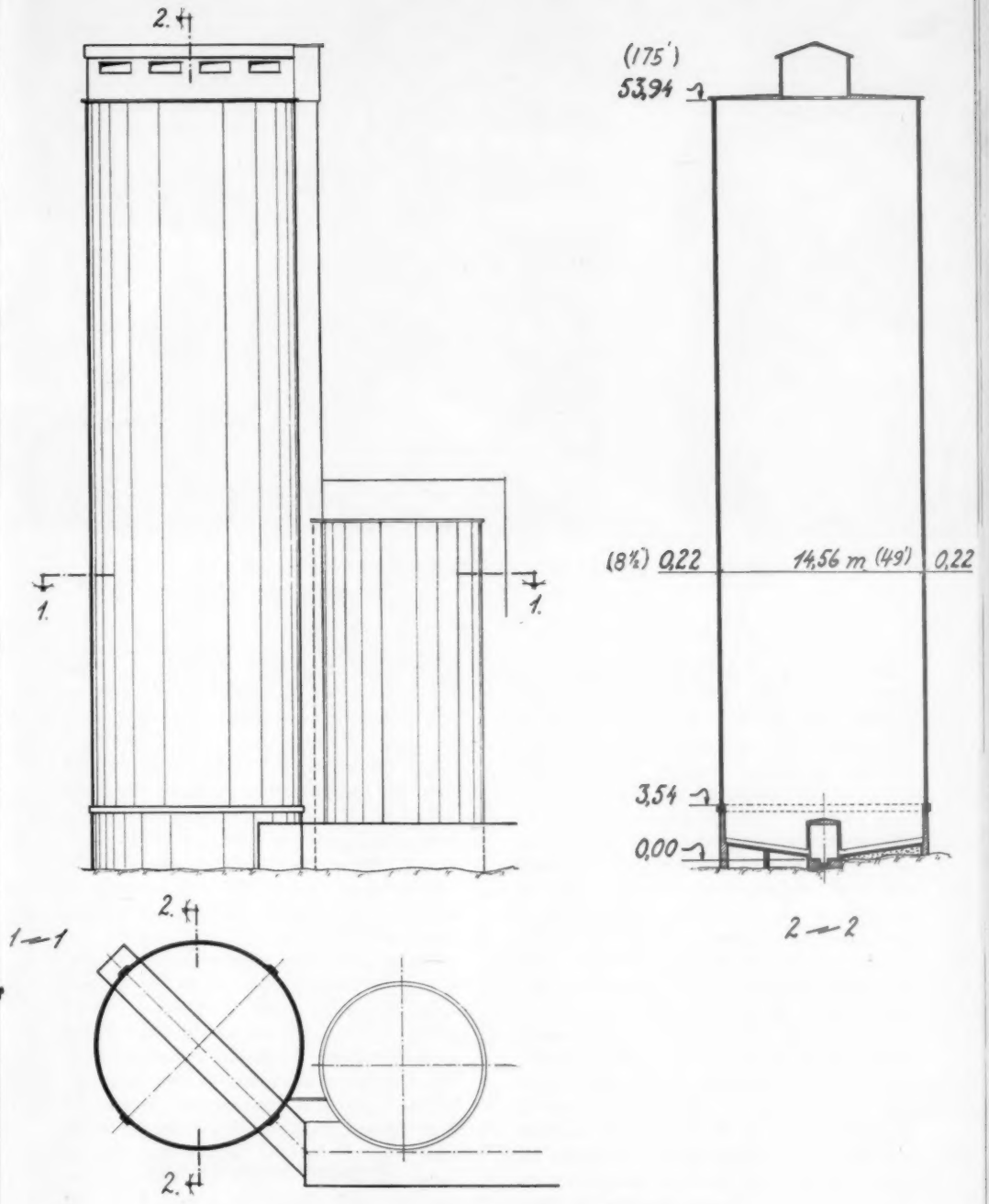


Figure 4. 175′ cement silos in Norway, cross-sections

Figure 5. Basin for 1.4 million gallons of cement slurry in Denmark.

after the sliding was finished (at this stage with a temporary timber support at the centre). This proceeding, which speeds up the construction and makes it more convenient, has proved economically successful for silos with diameters up to approx. 50'.

A total of some 20 prestressed cement silos have been constructed in Scandinavia and abroad during the last years, starting with a plant comprising 5 silos in Denmark in 1950. In all cases there has been taken into account a lateral pressure from the cement which is 50-100% higher than the active pressure which erroneously was used before the war. The introduction in the calculations of this increased pressure (known by geotechnicals as pressure at rest) has together with the prestressing completely relieved the silos of the cracking, which formerly was the weak point of any medium size silo. In this connection it should be mentioned that the same assumptions have been adapted to silos for storage of other kinds of granulated material (e.g. sugar).

In the greater part of the silos the cement arrives with a temperature of about 250° F, whereas the outer temperature easily could drop to 0° F. Theoretical investigations (which were confirmed by measurements) have shown, however, that the drop of temperature through the wall (which creates supplementary moments) will only be approximately 15% of the total difference of temperature, and this rather independently of the thickness of the wall. This is due to the insulating effect of the cement layers close to the wall in combination with the relatively high heat absorbing capacity of the latter.

Fig. 5 and 6 show another well-motivated use of prestressed concrete in the cement industry, viz. a basin for mixing of 1.4 million gallons of cement slurry, at the Danish Aalborg Portland Cement Factory. The economical advantage is here increased by the fact that the specific weight of slurry —and thus the thrust on the basin wall — is 60 to 80% higher than that of water. Moreover, cement slurry is — contrary to what should be expected — just as likely to go through cracks in the concrete as pure water.

The cement slurry is often stored in silo-like tanks (or standpipes) of rather small

Figure 6. Basin for 1.4 million gallons of cement slurry in Denmark, cross-sections.

Figure 7. Stand-pipes for cement slurry in Denmark.

diameter (20'-25'). Nevertheless prestressed concrete is indicated as the tensile forces in the wall are important at the base due to the high head (50'-60') of liquid with big specific weight. Fig. 7 shows such a plant at another Danish cement factory in the neighbourhood of Copenhagen.

Till now 4 basins and 8 standpipes in prestressed concrete for cement slurry have been constructed, mostly in Denmark.

SUGAR INDUSTRY

The construction of large sugar silos in prestressed concrete started in 1953 with a silo plant in Odense, holding 19,000 tons of white sugar in two silos. Fig. 8 and 9 show a more recent, but similar, Danish silo plant constructed in Goerlev for The Danish Sugar Company. It will be seen from fig. 8 that the Danish type of sugar silos, with interior diameter 66' and interior height 110' belongs to the simple tower-like type where the sugar is extracted by gravitation to a basement below the silo.

Fig. 10 shows the Swedish type of sugar silos which normally measures more in width than in height, as the sugar is extracted by means of a special machinery rotating around a central tower in steel. The sugar silos on fig. 10 with interior diameter 115' and interior height 90' hold each 22,000 tons of white sugar, and are probably the biggest existing silo units constructed in prestressed concrete, and with the use of slip forms. Fig. 11 shows these forms during the construction. In this connection it should be mentioned that the 1'-0" thick wall was provided with about 500 vertical ducts (diameter 3", distance approx. 1'), which were created during the construction by short sliding tubes, suspended to the slip forms.

About 12 silos of the Danish and Swedish type have up till now been constructed for a total storage of about 150,000 tons of sugar.

WATER STORAGE

This field of application for the prestressed concrete which is one of the most well-known shall only be illustrated by two examples.

Fig. 12 and 13 show one of two Swedish watertanks in Stockholm, each holding 4.5 million gallons of water, and thus being the biggest existing prestressed tanks in Europe. These tanks which are foundated directly on solid rock have been in service since 1954.

Figure 8. Silos for 19,000 tons of white sugar in Denmark.

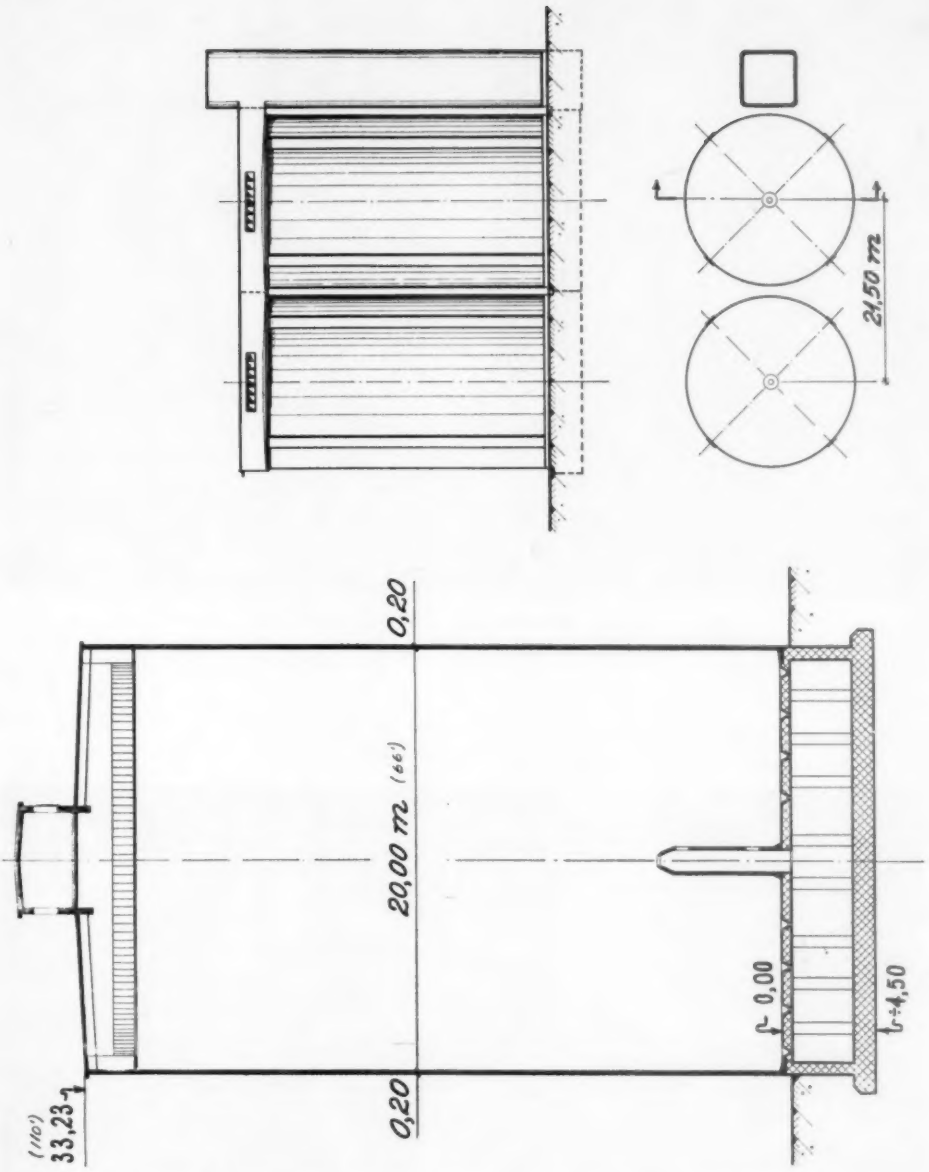


Figure 9. Silos for 19,000 tons of white sugar in Denmark, cross-sections.

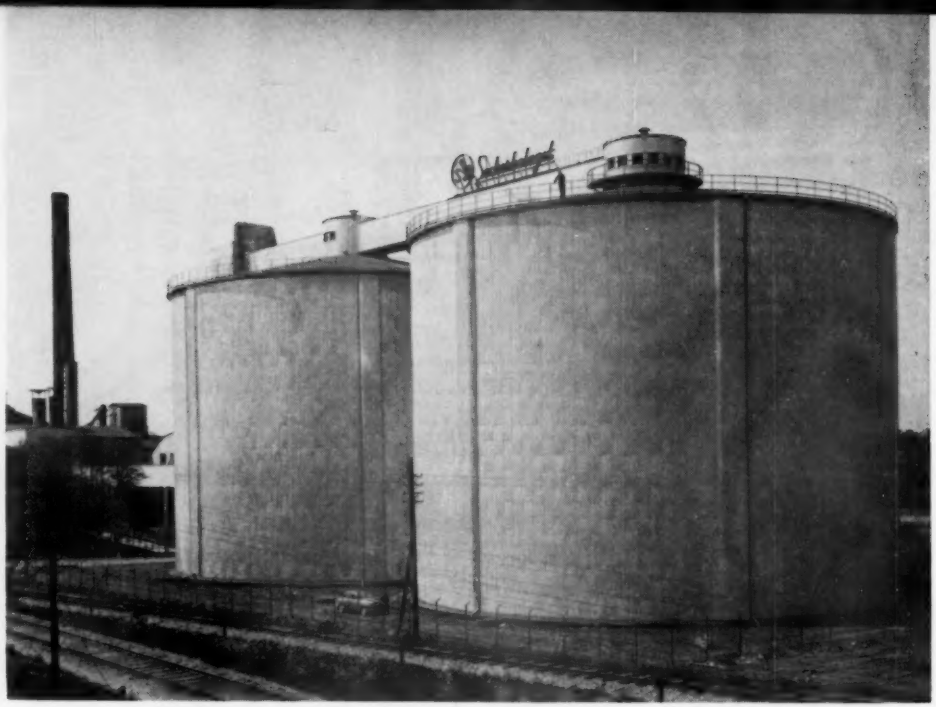


Figure 10. Silos for 44,000 tons of white sugar in Sweden.

•

Figure 11. Construction of one of the silos on fig. 10, with slip forms.

Fig. 14 shows a Danish watertower, actually (1959) under construction in Roskilde. The elevated tank holds 1.6 million gallons of water, and is constructed on the ground, after which it will be lifted by jacks to its final position on 6 columns about 70' above the ground level (this proceeding is silimar to that used for the water tower in Orebro, described in the October 1958 issue of Civil Engineering). An emergency reservoir at ground level, located between the columns will be accommodated to be used as swimming-pool (see fig. 14).

OIL STORAGE

The first Scandinavian attempt to store fuel-oil in prestressed concrete tanks was made in Denmark in 1948 with the construction of several small tanks, each holding about 150,000 gallons.

However, it was evident that larger units were required in the competition with steel tanks. In light of this Christiania Portland Cement Factory in Norway decided to build the tank shown on fig. 15 and 16, which holds 1.9 million gallons of fuel-oil, destined for rotating furnaces. It should be noted that the head of oil is very important, viz. 60', due to the limited available space for the tank; it has been in service nearly two years in an impeccable way.

Fig. 17 shows two tanks constructed for the Danish Aalborg Portland Cement Factory, each holding 2 million gallons of fuel-oil. As there was plenty of space available, the tanks were kept low (head of oil approx. 28') for economical reasons.

Finally fig. 18 shows the biggest European oil-tank constructed in prestressed concrete. This tanks which holds 3.1 million gallons of fuel-oil (interior diameter 105', head of oil approx. 50'), has been built for the Swedish Cement Corporation at two factories, in both cases with the use of slip forms.

The total number of tanks for oil-storage, constructed in accordance with the ideas indicated in this article is about 25. The greater part are tanks for heavy fuel-oil, nowadays constructed without interior lining, but they comprise tanks for gas-oil and crude-oil as well. The concrete tanks offer their owners considerable technical advantages, as such tanks may be partly buried without additional reinforcement, and as they to a large extent are fire-proof and impact-proof, costly safety-basins often can be avoided.

Figure 12. Interior of 4.5 million gallons water tank in Sweden.

FINAL REMARKS

The tanks and silos described in this article have been designed by the author's firm of consulting engineers, Chr. Ostenfeld & W. Jonson, Copenhagen. (The Swedish structures have been designed in collaboration with the consulting firm of Kjessler & Mannerstrale, Stockholm, whereas the Norwegian structures described in this article have been designed in collaboration with the consultants E. N. Hylland and Chr. F. Groner, Oslo.

The horizontal prestressing of all the structures has been done with Freyssinet-cables (12 wires 0.196" or 12 wires 0.276" according to the dimensions of the structure). The vertical prestressing has in some cases also been Freyssinet-cables, or — more commonly — 1" bars of the Dywidag system or the Swedish Halmstad system.

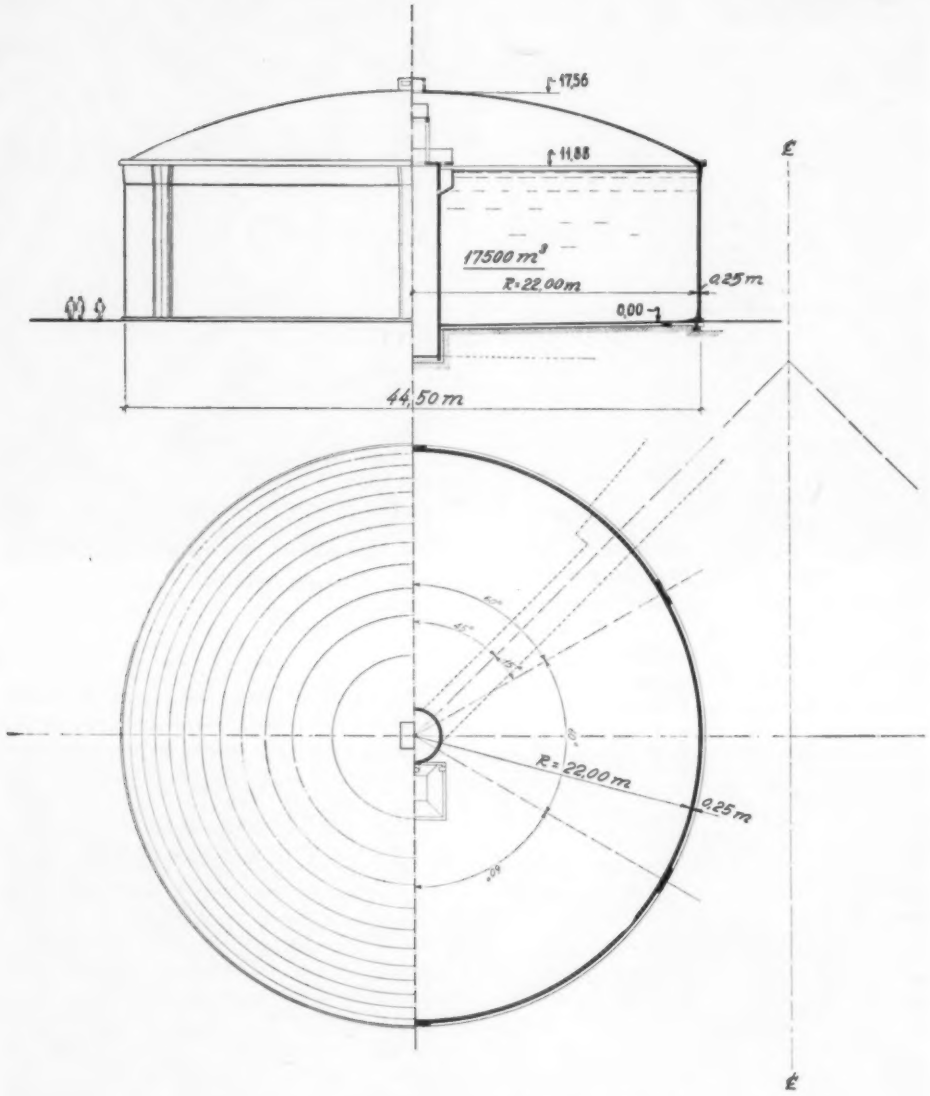
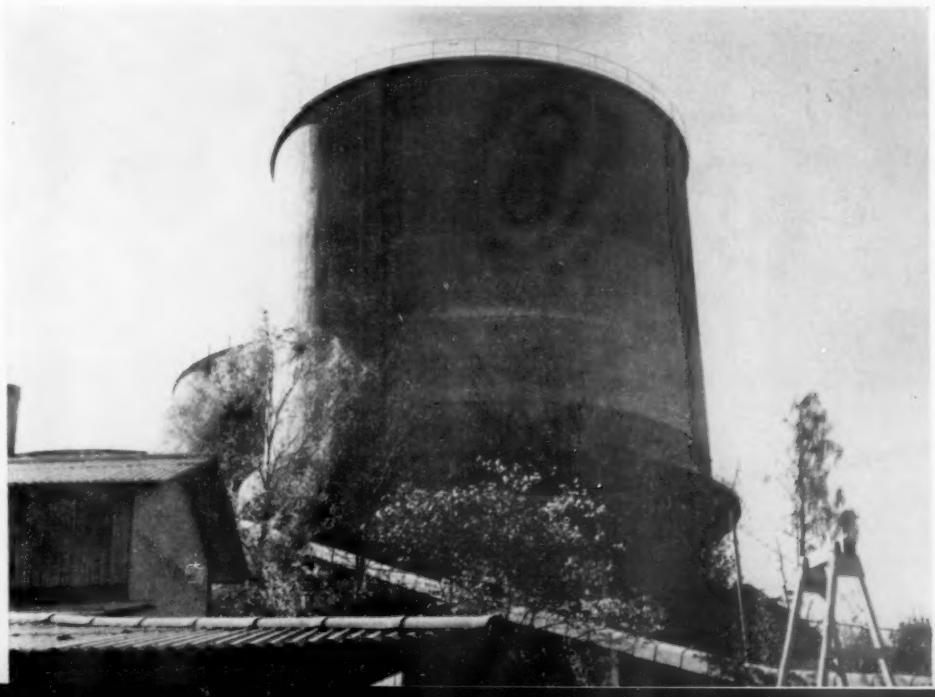


Figure 13. Interior of 4.5 million gallons water tank in Sweden, cross-sections.

Figure 14. One 1.6 million gallons water tower in Denmark (with underlying swimming-pool).

Figure 15. Tank for 1.9 million gallons fuel oil in Norway.



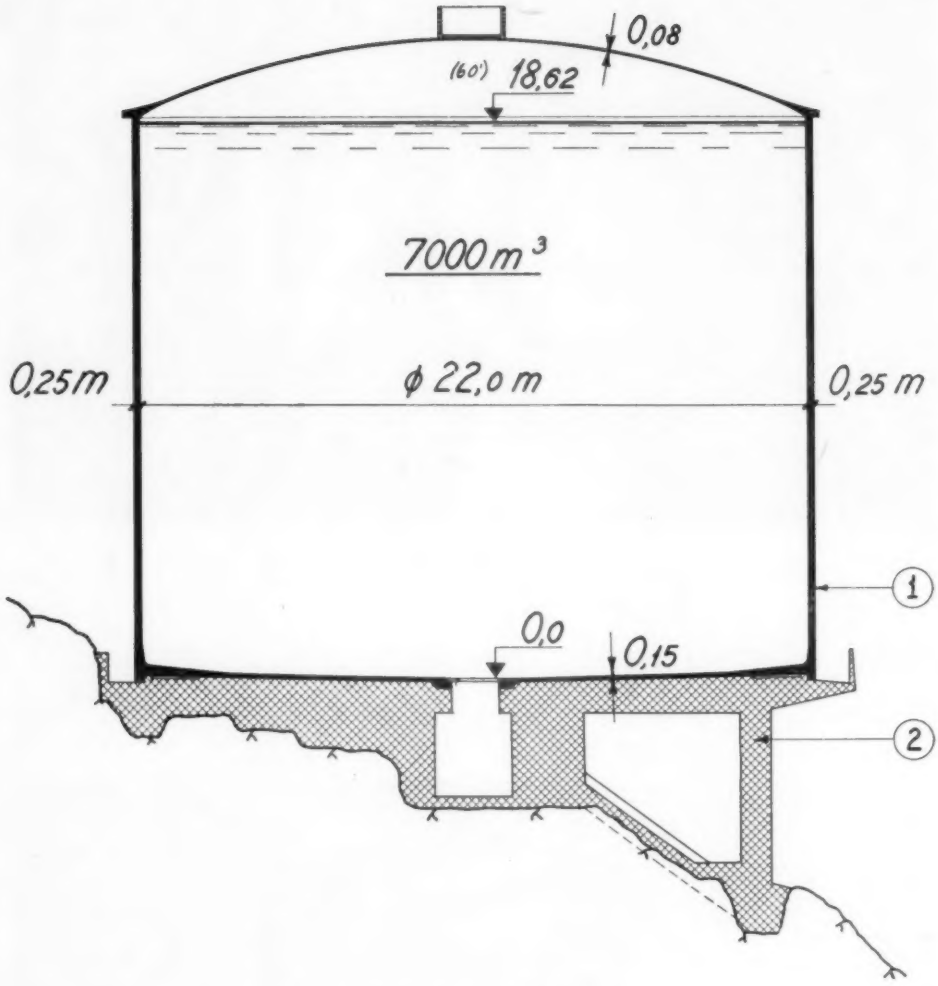


Figure 16. Tank for 1.9 million gallons fuel oil in Norway, cross-sections.

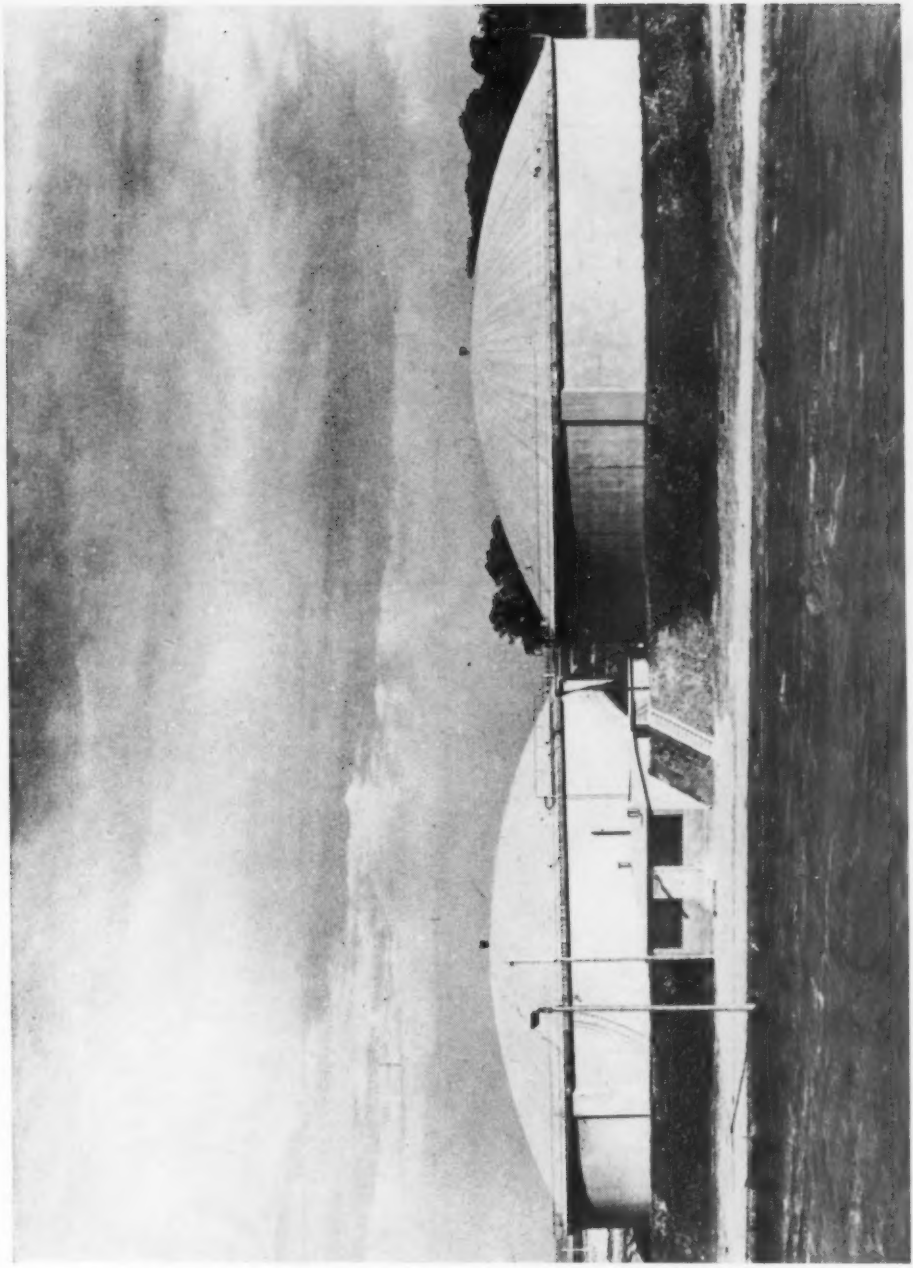


Figure 17. Tanks for 4 million gallons of fuel oil in Denmark.

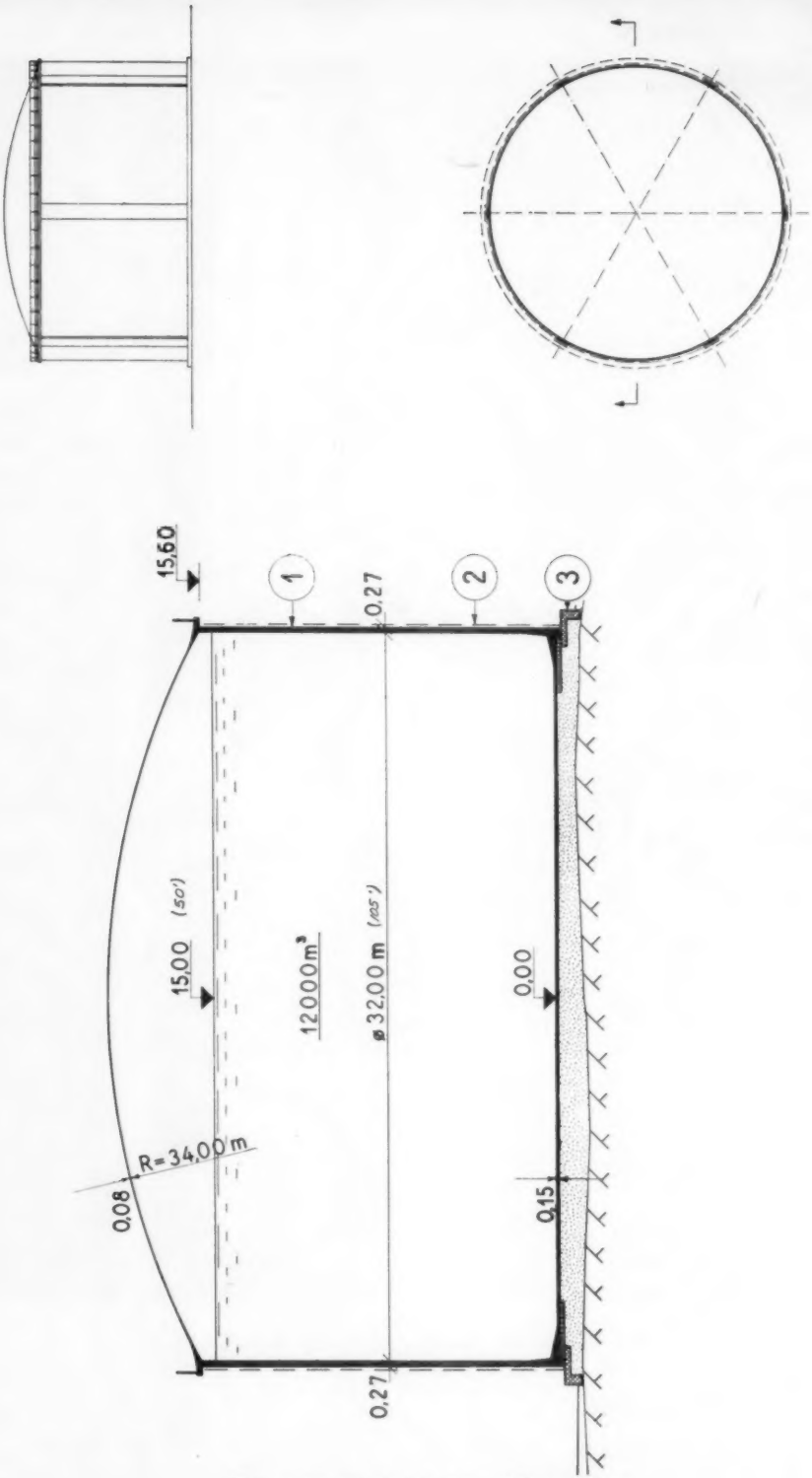


Figure 18. Tank for 3.1 million gallons of fuel oil in Sweden, cross-sections.

PARTIAL prestressing
and possibilities for its practical application

by

DR. P. W. ABELES
Consulting Engineer
London, England

1—The Essential Features of Prestressed Concrete

Prestressed concrete is unique inasmuch as it behaves like an elastic material when in compression or when the tensile resistance of the concrete is still available (i. e. before visible cracks have occurred). On cracking, a change in the behaviour takes place, as indicated in Fig. 1. This is a load deflection diagram of a beam which shows a sudden change in the rate of deformation at a certain loading, after this load is exceeded, the beam behaves similar to reinforced concrete with cracks developing and increasing in extent. The phenomenon of the behaviour of prestressed concrete which is of particular importance consists in the fact that full recovery takes place after the load has been reduced and the prestressing force has become again effective. The cracks close completely and the original slope of the load deflection curve is regained. This occurs not only after a single loading but after a million repetitions, as the author established as early as 1951. Only when the load reaches 80 to 90 per cent of the failure load, the cracks will not close, as a further change in the slope of the deflection curve occurs involving permanent deformations as indicated in Fig. 1. This example relates to an under-reinforced section (i.e. a section in which the steel is the weakest part and is of primary influence upon failure, when some yielding of the beam occurs). If, however, the section is over-reinforced (i.e. when the concrete compression zone is the weaker part and causes failure, before the proof-stress or yield point is reached in the steel) the third slope of the diagram does not occur and sudden brittle failure takes place, when the flexural compressive strength of the concrete is reached.

A further phenomenon of prestressed concrete may be mentioned; i.e. the development of very narrow microscopic cracks of a width of approximately 1/10,000 in. inside a section or at the surface. These cracks are not only very narrow but also very shallow and completely harmless. Such microscopic cracks were first discovered by Professor Evans of Leeds (1) and noticed by the author in 1951 and 1954 (2) and again in 1956 — 7 when many electric resistance strain gauges were affixed to the tensioned wires and to the outer concrete surface.

Normally, visible cracking occurs at the first break of the load-deflection diagram. However, by close observation there is no sudden break in a point but a gradual change as shown in the curved part of the enlarged portion of Fig. 1 and visible cracks can be observed when the plastic deformation is considerable. Some of the strain gauge readings showed that a sudden change of incline of the strain load diagram occurred already at the load when the straight line load deflection terminates (i.e. at a stress approximately the tensile strength or half the flexural strength) and a curved portion commences, as indicated in the enlargement of Fig. 1. From this it can be concluded that microscopic cracks of limited extent must have developed already at that stage, without however affecting the load at which such cracks become visible. The fatigue tests carried out by the British Railways have proved that these microscopic cracks did not adversely affect the load at which cracks became visible, even after one million cycles had been applied, during which these microscopic cracks must have opened and closed one million times.

Often the question is raised: When does a "microscopic" crack become "visible?" The author considers a satisfactory criterion in the condition that such microscopic cracks must not be noticeable above the lowest tensile edge of a beam when using a magnifier.

2—Two Ways of Approach

There are two ways of approach to prestressed concrete. One originated from the arch and has resulted in imparting to a construction, such as a beam, an artificial prestressing force of such magnitude that all cross sections remain always (i.e. during loading) under compression. In this connection one often speaks of "kernel" or "core" as well as of "pressure line", expressions which are closely linked with the arch. The famous French engineer Freyssinet applied prestressing to arch constructions early in this century and introduced it to other construction some 30 years ago by the creation of a "new homogeneous elastic material." This solution is being called "full" prestressing.

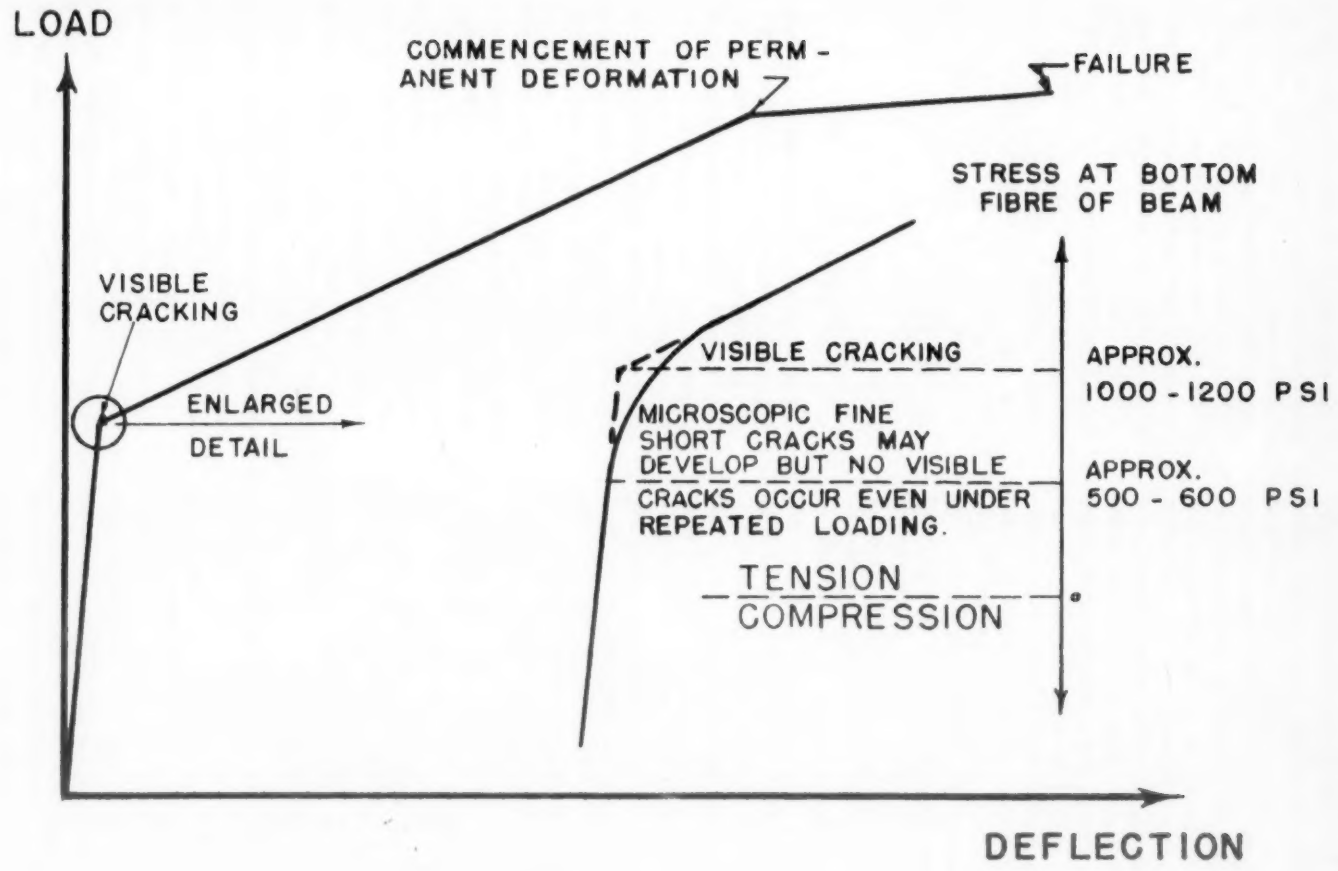

LOAD
COMMENCEMENT OF PERM-
ANENT DEFORMATION
FAILURE
STRESS AT BOTTOM
FIBRE OF BEAM
VISIBLE
CRACKING
ENLARGED
DETAIL
VISIBLE CRACKING
APPROX.
1000 - 1200 PSI
MICROSCOPIC FINE
SHORT CRACKS MAY
DEVELOP BUT NO VISIBLE
CRACKS OCCUR EVEN UNDER
REPEATED LOADING.
APPROX.
500 - 600 PSI
TENSION
COMPRESSION
DEFLECTION

FIG. 1

The other way of approach originated as a further development of ordinary reinforced concrete so as to allow the use of high strength materials to the full extent of their capacities and at the same time to exercise under working load control upon the extent of deformation and cracking so as to limit them to the permissible values. The author came into contact with such problems in connection with ordinary reinforced concrete some 25 years ago, when as an advisor of the Austrian Spun Poles Works he had to select the most suitable high strength steel reinforcement to obtain such conditions when designing for ultimate load. This was based on specific regulations for transmission poles set up by electrical engineers and not by reinforced concrete "specialists", who limited their considerations to an unrealistic nominal "elastic" design method for appreciable time. A steel material of a strength of 140,000 psi was selected by the author and comparative tests on rectangular concrete beams of different strength, reinforced with this high strength steel, indicated that the crack patterns were much better than those occurring with ordinary low strength reinforced concrete containing large diameter smooth mild steel bars, as the bond between the high strength concrete and the high strength steel bars of small diameters was excellent in spite of the much greater steel stresses. The only disadvantage of such a construction was the large deflection which indicated the desirability of a certain degree of prestressing. The first to suggest such a construction was the Austrian pioneer in reinforced concrete F. V. Emperger, an honorary member of the ACI. He proposed in 1939 the provision of additional pretensioned wires to reinforced concrete containing mild steel or work-hardened steel, as in use at that time in Europe, with the air to increase the permissible steel stresses by 50%. The author suggested in 1940 a wider use of this idea which he called "partial" prestressing to distinguish it from "full" prestressing, as only a part of the prestressing force required for full prestressing is to be applied in this case. This may be employed with pretensioning or with post-tensioning and either a part of the high strength steel may be tensioned to the maximum permissible stress or the entire steel may be tensioned to a lower initial stress so that the required effective prestressing force of definite magnitude is obtained. The entire steel reinforcement must according to the author's suggestion of 1940 be so great as to suffice for ultimate load conditions (3).

The expression "partial" prestressing which has been used for considerable time is perhaps not quite suitable since one might say that this term implies a solution at which only a part of the prestressing force is applied at one time and another part would be introduced at a later stage. Thus an expression "limited" prestress, i.e. involving the application of a limited prestressing force, would have been more appropriate. However, the term partial prestressing has been used already for such a long time that a change now would not be advisable.

3—The Philosophy of Design.

A new philosophy of design is gradually developing. The old idea of definite permissible working load stresses, without investigation of the factor of safety and mode of failure, are becoming obsolete, as they do not serve the practical requirements. Recent considerations have resulted in the demand that the following two conditions must be complied with in an adequate construction, i.e.

(1) A definite minimum load factor against failure must be available, the magnitude of which has to be adjusted to the specific circumstances and brittle sudden failure must be avoided under any circumstances; and (2) An adquate serviceability must be ensured under working load conditions resulting in limited maximum deformation and limitation of cracking.

With regard to ultimate load such considerations hardly occur with arch constructions and consequently originally with "full" prestressing investigations with regard to the conditions under ultimate load were ignored. Such an investigation is in fact unnecessary with full prestressing if (1) the section is under-reinforced, (2) the concrete cross section is not particularly small (which would, for example, be the case with double T-sections) and (3) the initial tensioning stress of the steel does not exceed 70% of the tensile strength of the steel. In such a case the required steel area to obtain suf-

ficiently large prestressing force that would ensure "full" prestressing will always be considerably greater than the cross sectional steel area required for the usual overall load factor of two. However, in the event of an over-reinforced section without special investigation it is obviously impossible to ascertain whether a section is over-reinforced or whether steel area is too small for ultimate load, sudden brittle failure will occur.

Another case at which such sudden brittle failure may occur is caused by diagonal shear due to combined shear and flexure near point loads after diagonal cracks have developed. This, for example, may occur when the tendons are solely based on working load conditions and are placed not closely enough to the outer tensile edge so as to prevent the development of heavy flexural cracks which then will soon turn into diagonal cracks when the load is increased, as the effective depth is greatly reduced. This would be the case with moving loads on simply supported beams, in which the tendons are located at the core, or with continuous beams in which the tendons may even be further away from the tensile edge, while with beams with steel placed closely to the tensile edge a much better crack distribution is obtained, as will be discussed more in detail later. Now it has generally been realized that ultimate load conditions must be considered at the design.

Another problem is the resistance to impact. It is well known that a completely elastic and brittle material such as glass or cast iron, is incapable of absorbing impact and will suddenly collapse. Brittle failure may occur even if the flexibility is only partially limited, thus great flexibility is of paramount importance in this atomic age, as will be discussed in more detail in Section 4, when considering four types of beams. It is a specific advantage of properly designed prestressed concrete that complete recovery will occur after such an impact loading.

With ordinary reinforced concrete appreciable cracking cannot be avoided under working load, and this applies first of all to cases at which smooth large diameter steel is used. The American reinforced concrete specialists were among the first to increase the bond conditions by the use of deformed bars and thus to improve the crack pattern by the replacement of a few wide cracks by many fine cracks. Generally, the development of cracks cannot be avoided in ordinary reinforced concrete because of shrinkage and the limited extensibility of the concrete. However, as long as these cracks are not too wide, they are harmless if the concrete is dense and weather resistant. In fact, corrosion of the steel and spalling off of the concrete will occur in humid surroundings, when the concrete is porous or not weather resistant in the event of change between frost and thaw, even if there are no cracks. Thus there is no need to require a greater limitation of cracking, as long as any possibility of stress corrosion in view of the high steel stress is avoided. This problem depends on a proper protection of the steel and it is essential to have it properly embedded in the concrete or grouted-in in the event of post-tensioning. Such provisions are much more important than the development of fine hair cracks. Another point is, of course, the question of magnitude of tensioning stress. In France for example, an initial stress of as high as 90% of the strength is propagated. Obviously the danger of stress corrosion becomes greater with increased steel stress under otherwise equal conditions.

It is certainly possible with prestressed concrete to limit the extent of cracking to any desirable extent. For certain constructions, particularly to those exposed to continuous impact and vibrations, such as turbo foundations, full homogeneity is very important. In other cases the development of occasional hair cracks under excess loads is completely harmless, provided that there are no cracks under ordinary condition. In other instances it will be quite satisfactory to allow the development of fine hair cracks under rare ordinary working load again if these cracks close completely under dead load. In certain circumstances it will even be of no harm if very fine hair cracks occur for considerable time, if the crack pattern is satisfactory.

The question of deformation is for prestressed concrete quite different from that occurring with ordinary reinforced concrete. In the latter case the main danger is that owing to creep, excessive deforma-

tion may occur in due course under working load. With prestressed concrete, however, it is an excessive camber which is most awkward. At the application of the prestress the concrete strength may be less than anticipated and the camber due to increased elastic deformation and following creep becomes much greater than expected. It is this excessive camber which has to be avoided rather than an excessive downward deformation under full working load as it occurs with ordinary reinforced concrete. Experience has proved that the downward deflection owing to load, after well distributed fine hair cracks have developed, is not excessive, as the strength of the concrete is much greater at that state.

Bearing in mind these ideas of a new philosophy of design, an experienced designer should be in a position (1) to select the required factor of safety against failure, appropriate to the conditions in question and (2) the permissible concrete tensile stress under working load, considering the extent and duration of the live load.

Technical and economical advantages are obtained by the reduction of the effective prestressing force required for full prestressing, if concrete tensile stresses are permitted under working load. These advantages consist in (1) a decrease of the construction depth or (2) an appreciable reduction of the initial camber or improvement of the economy or a combination of all of these advantages, and (3) increased flexibility and capacity of absorbing impact.

4—Types of Prestressed Structures.

There are two types of structures:

(a) a monolithic structure that has a definite tensile resistance, or a structure composed of precast members that has a definite tensile resistance at the mortar joints, and

(b) a structure in which the tensile strength of the concrete is not available. This may be due to shrinkage or other cracks which may have developed before prestressing, or due to joints between individual parts of the construction which are not or are insufficiently connected by cement mortar.

Provided that the maximum losses of the prestress are taken into account and that the actual maximum loading is in agreement with the design load, the condition that tensile stresses must not occur under design load results in entirely different factors of safety against cracking for the two types of structures in question.

With the Type (b), there is no safety against cracking and the factor becomes unity, whereas the Type (a) the factor of safety against cracking is substantial.

5—The Factor of Safety Against Cracking

As already shown in Section 2, a "fully" prestressed beam does not necessarily ensure safety against cracking unless guaranteed tensile resistance is available. However, it is generally assumed that such prestressed concrete structures, and particularly bridges, have an appreciable factor of safety against cracking.

In this connection it is interesting to note that in prestressed pipes which must be watertight only, a relatively small factor of safety against cracking is required. It was demonstrated at the early stage of development of prestressed pipes that temporary cracks which occur due to an overloading do not matter as long as the stress in the steel does not exceed its limit of elasticity. In fact, this phenomenon was used to illustrate the features of prestressed concrete pipes. Cracks which develop at such an overloading close completely afterwards and the pipe remains entirely watertight under operating pressure.

As already pointed out, extensive fatigue tests by British Railways have proved that freedom from cracks is ensured as long as the concrete tensile stress does not exceed approximately two-thirds of the modulus of rupture which is in good agreement with experience abroad. These tests have further proved that in the event of cracks having developed, they close completely on removal of the load, even after many millions of repetitions of loading, and the factor of safety against failure by static loading is not impaired by such a loading during which cracks open and close several million times. Thus a very small factor of safety against cracking will suffice to ensure greater flexibility.

6—Four Types of Beams.

As already stated in Section 1 prestressed concrete is a unique material and may ex-

hibit any type of behaviour when designed accordingly, i.e.

(a) elastic with brittle failure;

(b) elastic with slight plasticity before failure;

(c) elastic with considerable plasticity before failure, and

(d) plastic with slightly elastic behaviour at low stresses.

Figure 2

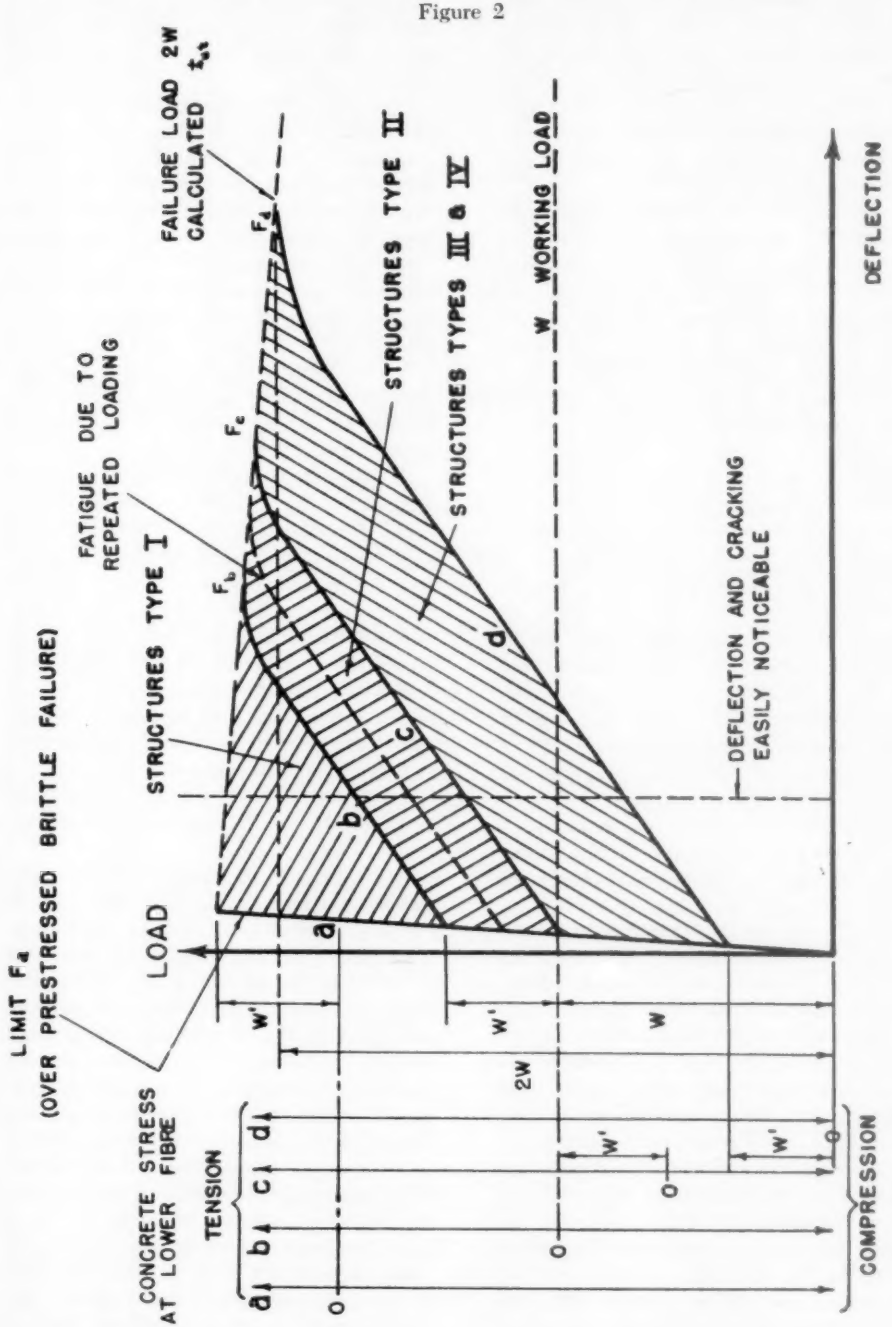


FIG. 2

In Figure 2, load deflection curves are shown for four types of structures considering under-reinforced sections with bonded wire for which the bending moment at failure depends solely upon the tensile reinforcement; they relate to beams of the same cross-section having the same steel area A_s and designed for the same working load W.

Beam (a) is over-prestressed, i.e. cracking and failure occur simultaneously as with a brittle material; case (d) relates to a non-prestressed beam while cases (b) and (c) are intermediate solutions.

In all instances the ultimate failure load W_u will be approximately the same. In the non-prestressed case (d), cracks will develop at a load W', when the tensile stress equals the flexural strength of the concrete (modulus of rupture). In beam (c), cracking is assumed to take place under the working load W; an effective pre-compression must therefore be ensured up to the loading W − W'.

Condition (b) represents the lower limit of "full" prestressing where permanent compressive stresses are ensured under working load.

In beam (a) there are compressive stresses up to the failure load less W' while in beam (b) the same conditions apply up to working load, in (c) up to the working load less W', in (d) there are no induced compressive stresses at all at any time and in (c) it corresponds to the limit where cracking would commence just when the working load is reached.

Type I are fully prestressed beams having limiting conditions (a) to (b), under which working load tensile stresses do not develop.

Type II are partially prestressed beams with limited tensile stresses at certain stages of loading, but ensure freedom from cracks under working load; this type is determined by limits "b" and "c". The lower limit "c" which corresponds to a zero point W − W' has to be adjusted according to the kind of loading to be applied; e.g. for fatigue loading instead of W', only 0.5 to 0.6 W' should be deducted from W. Again, for sustained loading, not more than 0.7 W' should be considered, while for rarely occurring loading this value may be raised to 0.9 W'.

Types III and IV are also partially prestressed, Type III being moderately prestressed in relation to the maximum working load which will rarely occur, Type IV slightly so, both types being determined by the limits "c" and "d". Type III is designed to be wholly under compression under ordinary loading and thus entirely free from cracks, while temporary fine cracks can develop under the rarely occurring full working load.

With Type IV beams, visible cracks may occur under ordinary loading.

Obviously beams Type II have a much greater resistance to shock than beams of Type I, the upper limit of which will suffer brittle failure, though all prestressed beams have resilience practically up to failure load, while non-prestressed beam (d) is not resilient though its shock resistance will be great.

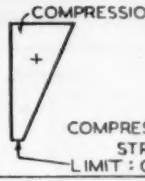
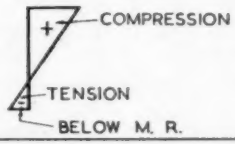
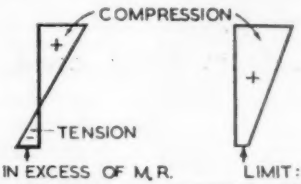
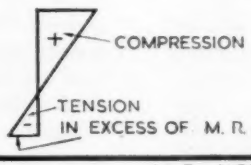
Beams Type II to IV will give an early warning of failure in the event of overloading as seen from Figure 1, in view of the noticeable deflections.

Of these four types, Type I is "fully" prestressed and Types II to IV are "partially" prestressed. These 4 types correspond to 3 types distinguished in the "First Report on Prestressed Concrete" prepared by the Institution of Structural Engineers, 1951, where Types I and II present type (i) in Figure 3, a structure in which freedom from cracks under working load is ensured. In particular Type I is equivalent to type (iA) of the "First Report" (fully prestressed) and Type II is equivalent to type (iB) in the "First Report." Types (ii) and (iii) correspond to Types III and IV respectively, discussed above.

The modulus of rupture corresponding to visible cracks of high strength concrete (28 days' cylinder strength approximately 6000 lb. per sq. in.) may be taken as 900 to 1000 psi. If psi is considered for pretensioning work and 900 psi for post-tensioning work, then bending tensile stresses of 750 and 670 psi respectively may be considered permissible for sustained loading of limited duration (e.g. 30 days) and stresses of 550 psi and 500 psi respectively for dynamic loading (fatigue conditions as with road bridges), provided that the range of loading is not too high. In all these cases freedom from cracks can be ensured.

Experience with Type II (corresponding to type (iB) of the "First Report") has been gained by British Railways, Eastern

THREE TYPES OF PRESTRESSED CONCRETE STRUCTURES

NO.	CHARACTERISTICS		WORKING LOAD STRESS	CONDITION	TYPE OF PRESTRESS
i	ALWAYS FREE FROM CRACKS	A		TRULY MONO-LITHIC	FULLY PRESTRESSED
				NON MONO-LITHIC	
		B		TRULY MONO-LITHIC	PARTIALLY PRESTRESSED
ii	TEMPORARY HAIRCRACKS UNDER RARE MAXIMUM WORKING LOAD FREE FROM CRACKS UNDER ORDINARY WORKING LOAD		(a) RARE MAXIMUM WORKING LOAD (b) ORDINARY WORKING LOAD 		
iii	FINE HAIR CRACKS UNDER WORKING LOAD. DEFLECTION CONTROLLED				PRESTRESSED REINFORCED HIGH STRENGTH CONCRETE

M. R. = MODULUS OF RUPTURE

Figure 3

Region since 1949. Recent inspections have proved that bridges and roof construction of this type are in very sound condition, although they were exposed to heavy smoke. The permissible tensile stresses under working load were for constructions with pretensioned wires 750 psi for static loading on roof constructions and 650 psi for abnormal dynamic loading at road bridges. For roof constructions with post-tensioned steel the permissible stresses were 600-650 psi. In all of these cases, another condition was to be fulfilled, i.e., that no tensile stresses should occur under dead weight, that in the event of an unexpected excess loading, and the development of cracks, the members would close up under dead weight.

7—The Basic Difference Between Full and Partial Prestressing.

The essential difference is that with full prestressing no concrete tensile stresses are to occur under working load while with partial prestressing substantial bending tensile stresses are allowed under working load, as seen from Figure 3. From the comparison it is obvious that with the same permissible compressive stress at transfer in both cases, resulting in the same effective prestress it is possible to obtain with partial prestressing a smaller section (and also a reduced prestressing force in view of the decreased section) than with full prestressing, as shown in Figure 3.

To repeat: A shallower construction depth

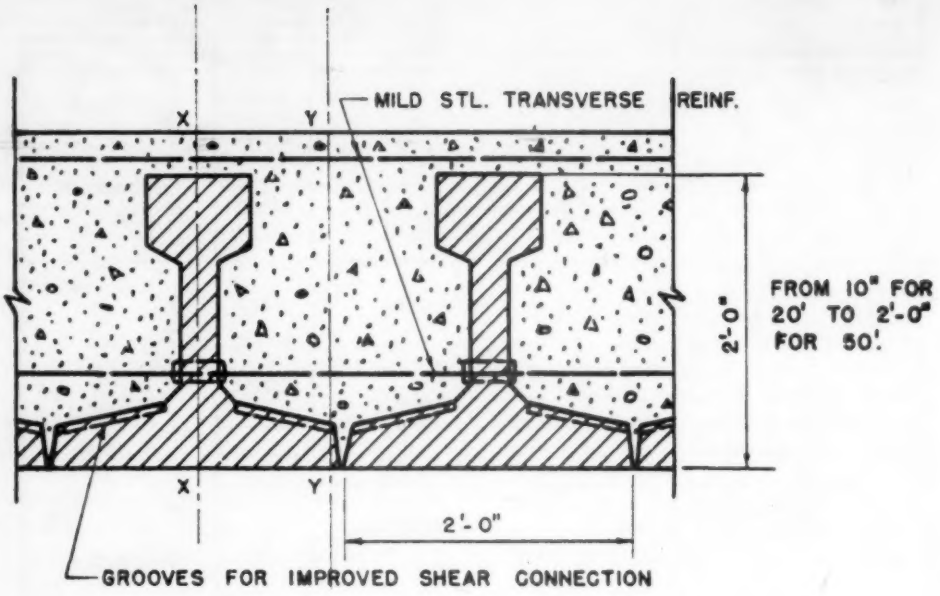


Figure 4

and reduced camber are mostly of technical advantage and a reduction in cross-section always means greater economy, while a decreased prestressing force results in saving anchorages and labour cost with post-tensioning. In most cases increased flexibility is essential to resist any impact.

8—The Practical Application of Partial Prestressing in Great Britain

The Chief Civil Engineer's Department, British Railways, Eastern Region, has introduced on the author's suggestion, a partially prestressed composite deck construction for high road bridges over the railway. They were required to replace old arch bridges so as to obtain sufficient headroom for overhead wires required for electrification of the railway. First a permissible tensile stress of 550 psi was allowed in 1948 and this stress was increased to 650 psi in 1952, after previously extensive static and fatigue tests had been carried out. This design ensures freedom from cracks under maximum working load and prestressed beams of varying depth have been standardized to be used for bridges of different spans between 20 and 50 feet.

Figure 4 shows a cross section of such a composite partially prestressed bridge deck used in the bridges mentioned before. The precast beams with pretensioned wires are designed to be as light as possible for economy and ease of handling; they are able to carry the adidtional concrete, and the composite section as a whole carries the additional load (road surfacing and live load). The same value for Young's modulus of elasticity can be assumed for both concretes, if the added concrete is well vibrated; the stresses due to difference in shrinkage and creep of the two concretes are small and may therefore be neglected. The composite deck acts as a monolithic slab since a good bond between the precast and in situ concrete is obtained. Consequently, the stresses can be superimposed as if the entire section were prestressed and the deformation of the added concrete is restrained by the precast concrete. In spite of these tensile stresses in the added concrete the development of visible cracks is prevented by the restraint due to the prestressed non-cracked precast outer tensile skin to which the in situ concrete is bonded.

Obviously, the maximum possible losses

of the prestress must be taken into account and strict supervision is essential to obtain agreement between the assumed effective prestress and the actual prestress in the structure. This can be successfully ascertained by acceptance tests.

Obviously the required factor of safety against failure had to be ensured and for the first bridges in 1948 additional non-tensioned mild steel was provided. After preliminary test with additional non-tensioned smooth high strength wires 0.2 in. dia. proved that full co-operation up to 90-100% of their strength was reached, such non-tensioned high strength wires have been used since 1949 to obtain the required steel area to ensure the specified factor of safety against failure.

Figure 5 shows placing of such beams at bridge constructions Fenchurch Street, London, and in Fig. 6 precast members stacked at a yard of a Prestressing Works are seen. It will be noticed that these are straight bottom and top wires and that special end blocks are not provided which were originally used, as might still be seen from Fig 5.

Partial prestressing has been employed in addition to bridges also for various roof constructions containing post-tensioned cables, for which a concrete tensile stress of 650 psi has been permitted, whereas for precast members with pretensioned steel the corresponding stress amounts to 750 psi.

Figures 7 and 8 show an example of cantilever roof beams of 120 ft. length employed for the roof of the Engine Shed at Ipswich, England. It will be noticed that the tendons were provided at both sides of the web and bent up from the tensile zone of the beam to the tensile zone of the Cantilever, the widening of the web being clearly seen in Fig. 7. This was done to allow good consolidation of the concrete in the web.

The conditions of various partially prestressed concrete constructions were investigated in 1956 (4) and these examinations were very satisfactory. The roof constructions were designed for a live load of 15 lb/ft^2 for snow and wind, because they are not accessible.

It is true that in England snow occurs rather intermittently, nevertheless this must be considered since full snow load may

Figure 5: Bridge construction, Fenchurch Street, London

take place for a considerable time occasionally. For example, in the winter of 1955/56, snow remained one to two months on most of the roofs. With regard to the roof at Ipswich (Figs. 7 and 8) the following may be said:

Figure 6: Precast members stockpiled in yard of producer.

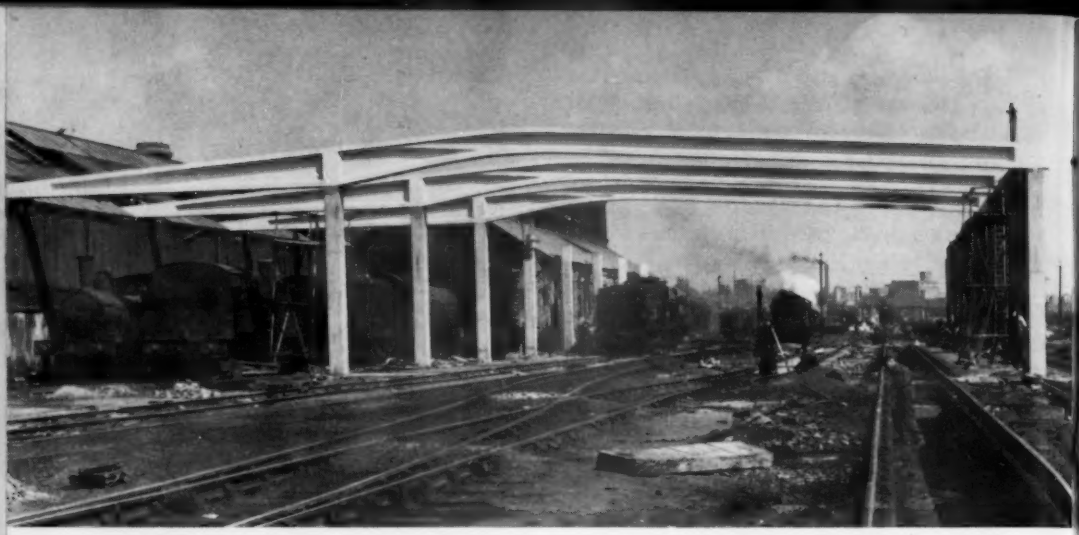


Figure 7: 120 ft. long cantilevered beams for roof of Engine Shed, Ipswich, England.

In some quarters it is thought that a particularly high factor of safety against cracking would be necessary where heavy smoke occurs, such as in an Engine Shed. However, any snow will immediately melt as long as the engine shed is in use and thus little such live load will occur. On the other hand, snow may remain on the roof when the engine shed is temporarily out of use without causing cracks.

Another example relates to the roof construction of a 4-bay building of a cleaning shed for electric engines, recently built at Ilford, Essex, near London, and illustrated in Figs. 9-11. The spans vary between 60 ft. 6 in. and 66 ft. 6 in. and the individual beams of 3 ft. depth contain in the bottom flange 42 pretensioned wires 0.276 in diameter and in the top flange 8 such wires. The flanges are 1 ft. 4 inches wide and 5 inches deep with 2½ inches chamfer and the web is 4 inches wide. These beams are designed for the span of 60 ft. 6 in. allowing for a permissible tensile stress of 750 psi under working load. For larger spans curved additional post-tensioned cables of 6 and 12 wires 0.276 in diameter are provided. At the supports these beams

Figure 8: Engine Shed roof, Ipswich, England.

are connected by both to upstanding parts of the columns so as to obtain some resistance in transverse direction. However, these bolts are only hand-tight and do not represent reinforcement for continuity as post-tensioning of continuity cables in the set-up construction was considered to be unsuitable for the job because of the complication of work which would have been involved at the site. Thus the beams are simply supported in vertical direction but form with columns four bay frames with 5 supports fixed at the bottom end and hinged at the top of the columns. This connection is obtained by the provision of steel bearing plates provided at the bottom ends of the beams and at the top side of the brackets of the column which plates were welded together at a specified medium temperature. Thus, this 4-bay frame structure is capable of transmitting horizontal wind pressure from the ends to all 5 columns.

The columns were originally considered in reinforced concrete of a cross section of 18 in. by 18 in. However, an alternative solution in prestressed concrete proved to be more economical. The precast columns had to be designed for reduced permissible stresses in view of their slenderness; the entire height below the brackets is 27 ft. and that above rail-level is 21 ft. 2 in. The alternative in prestressed concrete employed in the final construction is of I-shaped section of the same overall dimensions with flanges 3½ in. thick with 1 in. chamfer and having a reduction in cost to approximately two thirds of that of precast reinforced concrete was achieved by an appreciable reduction in steel reinforcement which consists of 8 pretensioned wires 0.2 in. dia. and 3 Tentor bars ½ in. dia. in each flange of the columns. Full size tests have proved the full adequacy of this design and its capacity to resist eccentric loadings of much greater magnitude. These columns were inserted into cavities provided in foundation, and they were then grouted in. The outside columns differed by the fact that brackets were provided only at one side. At the two transverse ends of the building pairs of concrete frames were provided to take up the wind pressure exerted at the transverse ends of the roof, as seen in Fig. 10.

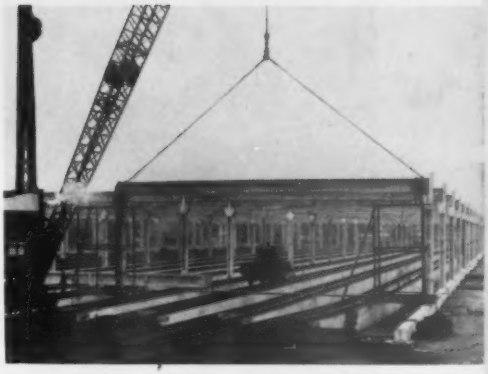


Figure 9: Placing beam, cleaning shed, Ilford, Essex.

Figure 10: Roof construction of cleaning shed, Ilford, Essex, showing light steel purlins.

Figure 11: Completed roof with skylights above each track.

Figure 12: Roof construction, Carolina Tractor Co., Charlotte, N. C.

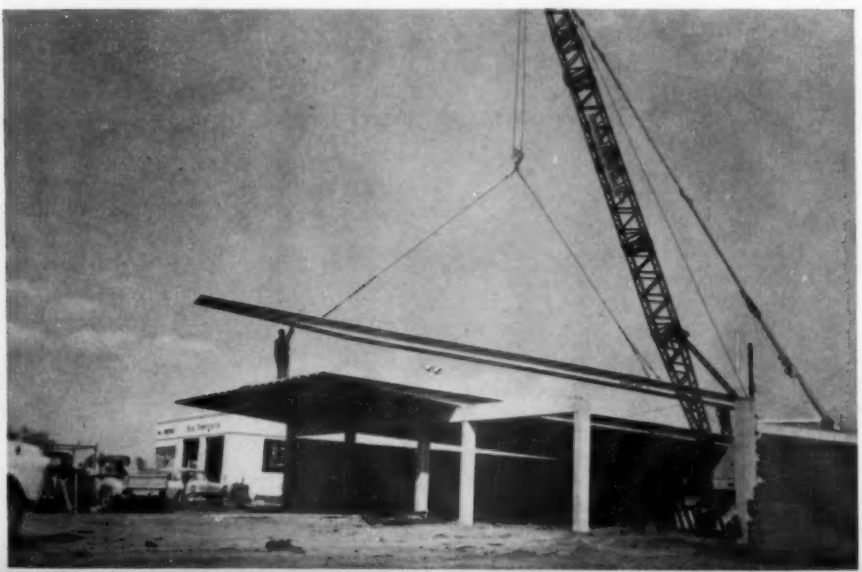
Fig. 9 shows placing of an individual beam which was accomplished quickly. In Fig. 10, the roof construction with light steel purlins is seen, while Fig. 11 illustrates the completed roof with sky lights above each track.

All these applications refer to structures of Type II in which freedom from cracks is insured though the factor of safety against cracking is relatively small.

9—Application in the United States

In recent years also in the United States partial prestressing has been introduced but mainly for the purpose of reducing the prestressing force and thus to decrease the camber. This is a very wise way of approach. The practical application of partial prestressing is not encouraged by the "Tentative Recommendations for Prestressed Concrete" of 1958, although the British "First Report on Prestressed Concrete of 1951" made such construction possible in Great Britain. In spite of this difficulty a great progress occurred in this country already in 1956 and was further developed in these last years. In fact, partial prestressing is being used here to a much

Figure 13: Roof construction, Drive-in Laundry, Charlotte, N. C.



more advanced degree than abroad.

In Great Britain and Germany generally only Type II ensuring freedom from cracks under working load is employed except for masts for which Type III has been introduced in these two countries. It is interesting to note that this Type III has been introduced here for roof and floor construction in such a way that freedom from visible cracks is ensured under dead load by providing effective compressive stresses under the dead load with the limit of zero stress at the tensile edge. This ensures reduced camber at transfer and freedom from cracks under dead weight.

The author was very glad to visit the plant of the President of the Prestressed Concrete Institute in February 1959 and thus to have an opportunity of seeing the pioneering work which he has introduced in Charlotte and surrounding by successfully using partially prestressed concrete of Type II which so far the author has not been able to apply to roof and floor construction elsewhere.

Fig. 12-14 show practical application of this type of construction. Fig. 12 is a roof construction of Carolina Tractor built in 1956 in Charlotte, N. C., and consists of 35 ft. craneway beams and 20 ft. roof double T beams. Fig. 13 shows the roof of a Drive-in Laundry, Charlotte, N. C., with partially prestressed beams of 50 ft. span and 15 ft. cantilever built in 1958.

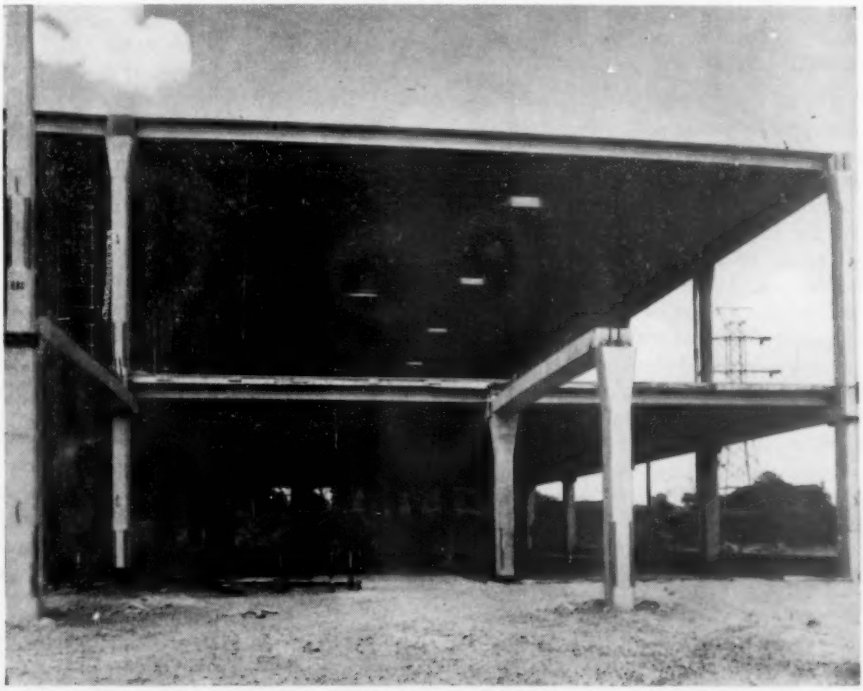
Fig. 14 illustrates the floor and roof construction of the Allian-Erwin Warehouse, Charlotte, N. C., having a floor span of 27 ft. and a roof span of 54 ft. builtin.

Partial prestressing of this kind has also been employed by other designers and reference may only be made to the imposing concrete roof for the Prestressed Concrete Roofs for Grandstands in Caracas, Venezuela, designed by T. Y. Lin and Associates.

9—Conclusions

Partial prestressing, when first suggested by the author in 1940 (3), was generally considered an inadequate solution, because it did not comply with the condition of a homogeneous material that always remains in compression. In the meantime, however, the creator of this idea, M. E. Freyssinet, himself has said in his autobiography (5): "In a road bridge, however, where there is not one chance in a thousand of the most unfavourable load being applied twice in

Figure 14: Floor and roof construction, Allian-Erwin warehouse, Charlotte, N. C.



the life of the structure, there is no disadvantage in allowing tensions of the order of 50kg/cm². (710 psi) in a concrete whose strains are appropriately guided; **to forbid it is a pure waste of public money.**" The extensive fatigue tests carried out by British Railways (2) have proved that millions of repetitions of loadings are harmless, even after visible hair cracks have developed which open and close million times. Thus, even if such most unfavourable, extraordinary loadings occur not only once but occasionally, with their cumulative effect they will hardly affect the life time of the structure.

Mr. Sadler, the Assistant Civil Engineer (Modernisation) has just described in a paper (6) the experience obtained by British Railways Eastern Region with partial prestressing during the last 10 years and expressed his view by saying "it is unfortunate that the introduction of tensile stresses in concrete arouses so much opposition amongst civil engineers." With British Railways, the main advantage of partial prestressing consists in reduction of depth and Mr. Sadler says "It will be appreciated that the railway civil engineer must take advantage of every facility and circumstance, **based on a sound economical design** which can be made available to him, so as to provide a greater headroom over the railway and yet at the same time preserve the level of roadway established by Act of Parliament."

While in the above case the reduction of depth is the main technical advantage, combined with enhanced economy, the production of precast concrete members without camber (or with greatly limited camber) is another important application of partial prestressing, as has been proved in the United States. A special advantage of this design is the limitation of the concrete compressive stresses at transfer. This reduces the danger of local failure at the time of prestressing, since the margin between minimum actual strength and applied compressive stress is greatly increased. Practical experience should convince engineers that a danger of local failure or even collapse of a structure may occur mainly due to excessive compressive stresses but never due to concrete tensile stresses, provided that a tensile reinforcement is available. What may happen in such a case, would be only the development of cracks, resulting in a warning of some fault in the construction that can be put right, but would not lead to a sudden collapse. How many anchorages have broken in at the moment of prestressing owing to insufficient strength of the concrete and how often did the concrete crush at stressing in structures composed of individual units with insufficiently strong joints?

Obviously, ultimate load conditions must be correctly investigated. However, this applies not only to partial but also to full prestressing. Shear failure is just as sudden as compression failure and should be avoided under any circumstances. This applies particularly to a combination of bending and shear in long beams, as the author has shown in (7). Due to diagonal cracks, failure may occur at relatively low shear stresses if the flexural tensile stresses are high and thus the extent of cracking unfavourable before the development of such diagonal cracks. Some experts are of the opinion that solely ultimate load conditions need be considered and that an investigation of the working load condition is only a formal requirement to satisfy those engineers who do not appreciate the advantages from partial prestressing. In that respect it is considered satisfactory by the experts mentioned above that only nominal losses are taken into account. However, the author cannot agree with these considerations. In his view it is important to establish the maximum possible losses of prestress appropriate to the case in question so as to obtain the minimum prestress which is expected to be effective and thus to know at which loading this prestress is just available. One is thus, more or less, certain that any hair crack which might have developed previously will be completely closed under such a loading that may, for example, correspond to the dead load or to any other specified loading. These maximum losses depend on the magnitude of the concrete stresses at transfer and under dead load at the centroid of the steel, on the strength properties of concrete and steel and on the percentage of humidity at the environment. The assumption of approximate average losses may lead to great misconceptions.

In addition to the two mentioned cases of appplication of partial prestressing i.e. (1) reduction in construction depth and (2) reduction in camber, there is a third possibility i.e. saving material. This condition is most likely less important for the United States than for other countries where it seems to be of great economical advantage to replace mild steel reinforcement by a smaller quantity of high strength steel. This development has gone very far in many countries and steel stresses of 50,000 psi and more are permitted in certain countries. Obviously, by the application of partial prestressing and the use of high strength prestressing steel it would be possible to reduce the steel area to 1/6 of that required for mild steel. The practical introduction of this idea has been advocated by Professor Zerna, Hannover (8) who says that the idea of partial prestressing allows the production of concrete structures of a quality equivalent to ordinary reinforced concrete in which reinforcing steel of any high strength can be fully utilized and thus technical and economical advantages be obtained. Obviously, first class workmanship is essential with such a type of partial prestressing. It is often said that it is impossible to rely on such first class work and that bad execution ought to be taken into account. However, such an approach is not applicable to prestressed concrete whether full or partial prestressing is employed, as certain effective prestressing forces and concrete properties must be available; otherwise the design would not be adequate. Hence, it is essential to ensure that in every individual instance the required properties are obtained and if this is achieved there is no justification why not full use should be made of these properties.

References

(1) R. H. Evans "Extensibility and Modulus of Rupture of Concrete", Structural Engineer, December 1946;

(2) P. W. Abeles "Static and Fatigue Tests on Partially Prestressed Concrete Constructions" J.A.C.I. Vol. 26, 4 (Dec. 1954);

(3) P. W. Abeles "Saving Steel by Prestressing", Concrete and Construct. Eng., July 1940;
"Fully and Partly Prestressed Reinforced Concrete" J.A.C.I. Vol. 16, 3 (Jan. 1945);

(4) P. W. Abeles "The Conditions of Partially Prestressed Concrete Structures After 3 to 7 Years' Use" Final Report 7th Congress Lisbon 1957, Int. Assoc.f. Bridge & Struct. Eng.

(5) E. Freyssinet "The Birth of Prestressing" Cement and Concrete Assoc., London, Transl. No. 59, 1956;

(6) R. E. Sadler "Development in Overhead Electrification of Railways as it Affects the Civil Engineer", Paper No. 6320, Proc. Inst. of Civil Eng., London, February 1959;

(7) P. W. Abeles "Diagonal Shear Failure in Prestressed Concrete Beams with Well-Bonded Steel" Third Congress F.I. de la Precontrainte, Berlin, 1958;

(8) W. Zerna "Teilweise Vorgespannter Beton" (Partially prestressed concrete), Beton und Stahlbeton, H.12, 1956.

One of our Active Members sends us this problem and asks for comments:

DEAR COL. KORN:

We have been making prestressed concrete piles for an important project near here and have encountered some difficulties in driving. Some of the piles have driven well, some of them have developed horizontal cracks around the piles at various points. Several were badly spalled at the top and in one case on an isolated pile a diagonal crack appeared. The design driving resistance was 80 blows to the inch but many were driven from 100 to 124 blows without distress.

The piles were square with a design prestress of 750 psi and were driven with a Vulcan O.R. hammer which has a rated capacity of 30 ft.-kips per blow.

We want to find out if heavy spalling is unusual. Have you had any reports which pointed to the hammer not hitting the pile squarely as the cause of spalling? Do you have any suggestions on what we might do to alleviate this spalling due to driving?

Reply by Col. Korn:

The pile design that you describe in your letter is very similar to many other prestressed pile designs. I assume from your letter that most of the trouble occurs at the driving head rather than distress further down on the pile, which might be attributed to eccentric vibrations on a long unsupported length.

Spalling of prestressed piles has been known to occur if the end of the pile is not perfectly square with the axis in both directions. Naturally this causes one edge to take all of the impact and the concrete flakes down to the spiral. Sometimes pile spalling is due to improper cushions or a failure to properly clean the driving cap of the old burned out material. If wood is used and it is allowed to accumulate around

the perimeter of the cap it can become virtually incompressible and consequently most of the energy will be transmitted to the pile at these points.

Perhaps you have already investigated the above factors. We will, in the meantime, endeavor to secure information on the subject from others of wide experience.

Comment follows by Active Members with extensive prestressed piling experience:

SIR . . .

We have experienced very little trouble with spalling at the head of the pile. When it has occurred, we have usually found the head of the pile to be out of square. In one particular case, however, we did discover improper cushions being used. Spalling ceased immediately when proper cushions were installed. Of course, the prestressing strands must be cut flush with the head of the pile. Slightly protruding strands, combined with ineffective cushioning, could cause spalling.

SIR . . .

We have had trouble on only one job with horizontal cracking and spalling other than at the head of the piles. Everything on this particular job was done wrong, including the hauling of the piles. The piles were 12" square and 65 feet long. They were hauled on a pole trailer and supported at the front end of the trailer and at the end of the pile with .21 of the length overhanging at the rear. Small horizontal cracks developed while being transported due to too much unsupported length between bolsters.

The piles were driven with a drop hammer mounted on a skid-rig driver. No water jets were used to assist the driving in very hard ground. When a pile started to run out of line, the crew attempted to pull it at the top and continue driving. This caused the horizontal hairline cracks to open and some spalling resulted. One pile was even broken in two.

It was interesting to note that when this very improper procedure was corrected, no further breakage or spalling occurred, even in piles which were slightly cracked in hauling.

SIR . . .

Fortunately, our problems with prestressed concrete piles have been few. We have furnished several hundred thousand feet with serious trouble on only the one small job.

SIR . . .

We would suggest that the following items be carefully checked:

1. Check the head of the pile for square.
2. Check the cushioning material.
3. Check the manner of handling at the plant, transporting, and the handling at the job.

EDITOR'S NOTE. See article on page 58 of this issue, "Torsion in Concrete Piles during Driving" by Ben C. Gerwick, Jr.

EFFECT OF TEMPERATURE DROP ON STRAND STRESSES IN A CASTING BED

H. KENT PRESTON, Engineer
JOHN A. ROEBLING'S SONS CORPORATION

One of the problems that has been encountered in the fabrication of pre-tensioned prestressed concrete members has been that of the effect of temperature change during the curing period and before the strands have been unjacked. In a few instances the combination of conditions has been so unfavorable that all the strands have broken at the temporary grips when a temperature drop of appreciable magnitude has occurred. The following data are presented to help fabricators anticipate and avoid similar failures.

A study of the conditions shows that danger of strand failure exists when the length of exposed strand (strand not encased in a concrete member) is small with respect to the total length of concrete members in the bed.

Since the strands are a fixed length, being attached to immovable anchors at each end, their tension goes up as the temperature drops. If the concrete members were a fixed length they, too, would develop a tension in proportion to the temperature drop. Actually they are only partially restrained by the strands and therefore shorten some but not so much as if completely unrestrained. When the concrete shortens the exposed strands must lengthen an equal amount. As a result there is an increase in tension in the concrete and the strands embedded therein, which is less than proportional to the temperature drop while the exposed strands undergo a tension increase larger than that proportional to the temperature drop.

The analysis of this condition **is necessarily approximate** since one of the more important factors in the calculations, the modulus of elasticity of the concrete members, will depend upon the age and strength of the concrete at the time of the temperature drop. Another approximation is the assumption that the temperature of the strand drops as much as the air temperature, while the temperature of the concrete and the strands embedded therein drops only two-thirds as much because of the mass of the concrete and the relatively short duration of the low temperature. Friction of the concrete members on their pallets will tend to resist the shortening motion in the concrete and will therefore have the same effect as a smaller temperature drop. This factor is included in the assumption that the temperature of the concrete drops only two-thirds as much as the strand.

The curves in Figure 1 cannot be taken as exact but do show when the combination of temperature drop plus concrete and strand lengths is in the danger zone. These curves are based on the following assumptions:

Modulus of Elasticity of Concrete, E_c at time of temperature drop is 2,8000,000 p.s.i. (Equivalent to an ultimate strength of 2000 to 2500 p.s.i.)

For Curve No. 1

Tension in strand = 85% of catalog ultimate strength. Strands sometimes break in the temporary grips at this load and if one strand breaks the load in the other is increased, which tends to make them all break.

For Curve No. 2

Tension in strand = 90% of catalog ultimate strength.

In the member considered the cross-sectional area of the concrete is 175 times the area of the strands.

When interpreting the curve, four factors should be kept in mind:

1. If the area of the concrete is increased with respect to the area of the strands, the temperature drop required to raise the tension to that specified will be less than that indicated by the curve. The opposite is true if the area of the strand is increased.
2. If the concrete strength and modulus of elasticity at the time of the temperature drop are greater than assumed herein, the temperature drop required to raise the strand tension to 85% or 90% of ultimate will be less than that shown by the curve.
3. When stress increase due to temperature drop is a factor, proper location of the concrete members in the bed is important. The length of exposed strand at the jacking end of the bed should equal the length of exposed strand at the anchor end and the spaces between the concrete members should be the minimum possible. The curve is based on the assumption that this arrangement has been followed.
4. If the temporary grips could develop the catalog ultimate strength of the strand there would be no problem because of the large elongation that takes place between 85% and 100% of the ultimate.

A typical example of the use of the curves in finding the approximate temperature drop which will create a given tension is shown in Figure 2.

When the combination of conditions indicates possible trouble, one logical precautionary measure would be covering both the concrete members and the exposed strands to minimize the temperature drop.

EXAMPLE OF USE OF CURVES SHOWN IN FIGURE 1

See Figure 2 for dimensions

Measure values L_{s_1}, L_{s_2}, L_{c_1}, L_{c_2}, etc. from bed and add. Assume these measurements give --

$$L_{c_1}, + L_{c_2} + L_{c_3} = 400.0 \text{ feet} = L_c$$

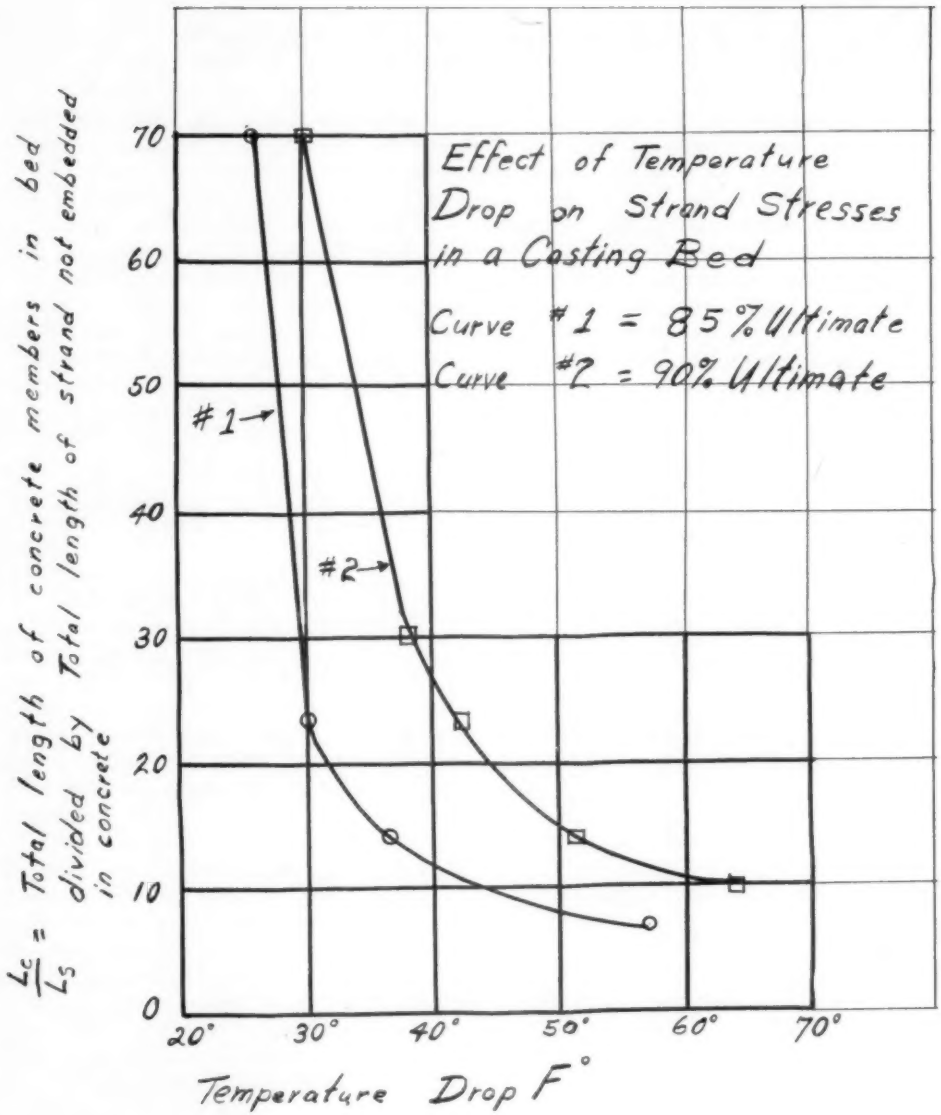
$$L_{s_1}, + L_{s_2} + L_{s_3} + L_{s_4} = 400 \text{ inches} = L_s$$

$$\text{Then } L_c/L_s = \frac{400' \times 12''}{400''} = 12$$

Allow strand load of 85% of ultimate which means use Curve #1.

From Curve #1 at $L_c/L_s = 12$ we read 40° F.

Thus for this set-up and the conditions outlined in the text a temperature drop of 40°F would raise the strand tension to 85% of catalog ultimate.


Effect of Temperature Drop on Strand Stresses in a Casting Bed
Curve #1 = 85% Ultimate
Curve #2 = 90% Ultimate
#1
#2
$\frac{L_c}{L_s}$ = Total length of concrete members in bed divided by Total length of strand not embedded in concrete
0
10
20
30
40
50
60
70
20°
30°
40°
50°
60°
70°
Temperature Drop F°

Arrangement of Members in Bed

TORSION IN CONCRETE PILES DURING DRIVING

BEN C. GERWICK, JR.

Ben C. Gerwick, Inc.
San Francisco, Cal.

On three recent widely separated projects for the installation of concrete piles, cracking occurred on a number of piles during driving. The cause was initially inexplicable but careful field investigation indicates it was due to a combination of torsion stresses with pile hammer rebound (tension) stresses. These torsion stresses were produced by the hammer imparting a twisting impact blow to the head of the piles; the twisting action being resisted by the inertia of the pile and the soil into which it was driven. While the three cases in question occurred with prestressed concrete piles, the circumstances and symptoms are very similar to past difficulties with conventionally reinforced concrete piles, which at the time were conveniently labeled as due to "too hard driving."

This torsion cracking can be eliminated by proper driving equipment and techniques. The practical solution is very simple. However, an exact explanation of the combination stresses occurring will require further research and laboratory study. This paper is purposely being published prior to a full theoretical understanding in order to eliminate further difficulties in the field and as a stimulus to research into the problem.

The first instance occurred with prestressed concrete piles being driven in a large wharf project. The piles were octagonal in cross section and were extremely strong and well-made. They were driven with a heavy steam hammer mounted in a modern steel universal piledriver. Horizontal cracks appeared during driving in about one out of every three piles. Variations in hammer size and driving techniques failed to solve the problem. Therefore, a thorough investigation was initiated.

The cracks occurred without warning and without apparent cause. They were always similar and always progressed to failure if driving was continued. There was no apparent relationship to hard driving as one pile took driving up to 1000 blows per foot without any signs of distress. Also, there was no apparent relation to bending stresses during driving. Many of the piles would "jump 3 to 6" when the hammer and driving head was lifted off the pile at the conclusion of driving, indicating heavy bending stresses, but these did not produce any cracks.

The symptoms were as follows: At some period during the driving, sometimes after only 40 blows or so, a horizontal crack would occur completely around the pile, about 3/10 of the way down from the head of the pile. The crack would not close even if driving was stopped immediately. Under subsequent blows, this crack would begin to puff concrete dust, then open wider, and spall flakes from the surface. If driving was continued further, similar cracks would occur at about 2 foot intervals above and below the first. If driving continued, the pile would eventually fail completely with an inclined break.

All piles exhibiting the slightest crack or puff of dust were jetted free and pulled and replaced.

One of the disconcerting things about this phenomenon was that 10 to 20 piles could be driven without any difficulty whatsoever, then several piles in a row would develop cracks, under what appeared to be identical driving conditions and methods.

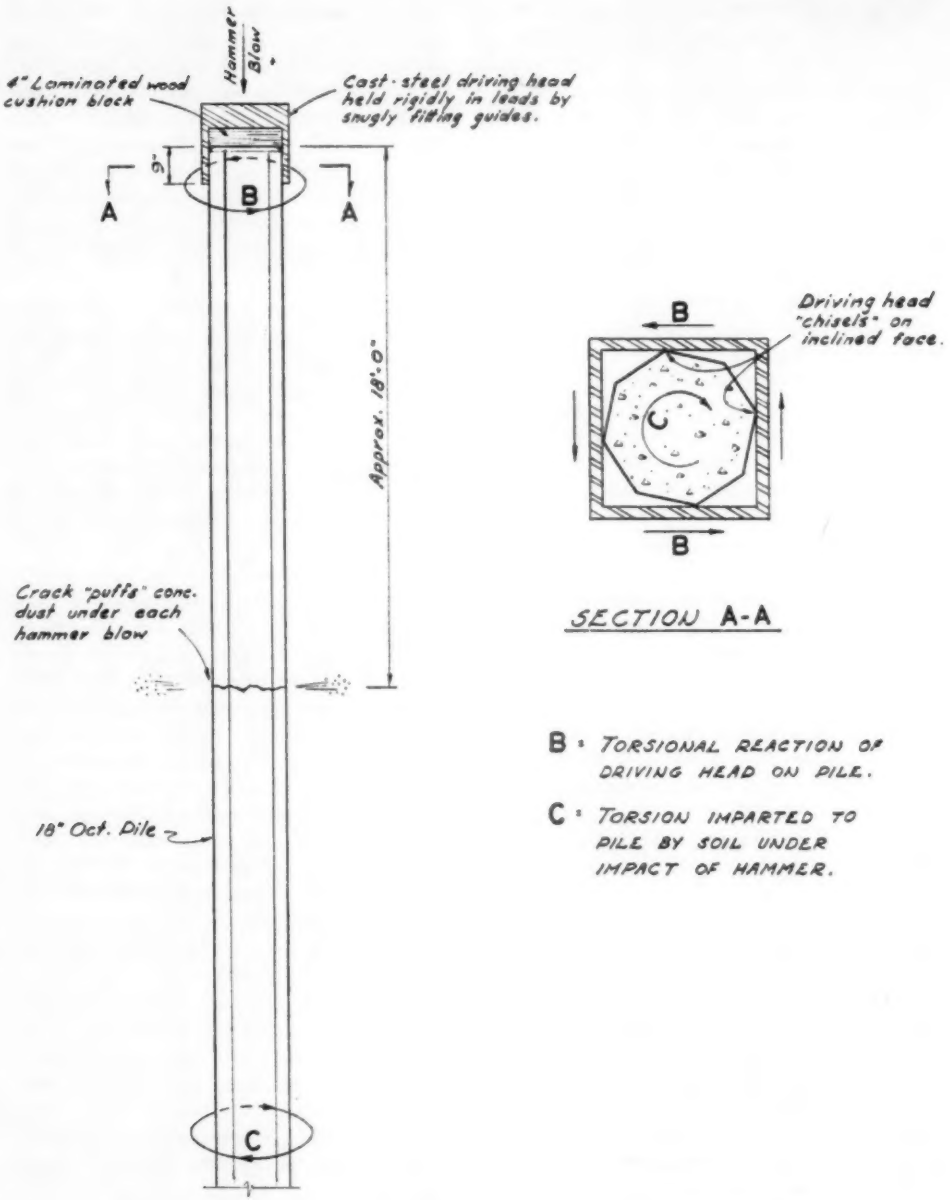
Thorough checks were made of the manufacturing process and the completed piles. All appeared excellent. Therefore, a concentrated study was made of the driving equipment and techniques to see what, if anything, was different from those which had been employed successfully in the past.

The driver was a modern steel universal driver with heavy, rigid, leads. Between the hammer and the pile was a driving head (helmet) to which was attached a set of steel channel guides, called a jig, which rode up and down in the leads, thus holding the driving head quite firmly. The floating driver was held accurately in position with 5 taut steel anchor lines to heavy anchors. The driving head, made of cast steel, fitted the head of the pile snugly, and was 14" deep. A cushion block of 5 laminations of 1" Douglas Fir was placed on the head of the pile: however, this compressed to 3" or less under the first blow of the hammer. Thus the driving head had a snug fit for the top 12" or so of the pile.

The material into which the pile was being driven consisted of sand and partially cemented shell-sand conglomerate, and was very firm and dense with, however, small voids in the shell fragments that allowed the pile to penetrate under driving. It gripped the pile firmly, more or less cementing itself to the sides of the piles, and locking them in position.

The deep recess in the driving head was the first thing noticed as being out of the ordinary and different from past practice. So as a trial, the head was filled so that the recess was only about 4" deep. This gave immediate improvement, and more than seventy piles were driven without cracking.

Meanwhile, investigation was continued in an attempt to find the cause. A careful inspection was made of the piles which had cracked and been pulled and rejected. The typical crack, short of complete failure, was quite wide (about 1/16") and went all around the pile horizontally. The surface of the pile adjacent to the crack was



SCHEMATIC DIAGRAM SHOWING HOW DRIVING HEAD IMPARTS TORSION TO PILES

Figure 1

severely spalled. Chipping into the crack revealed the rather astounding fact that the crack was only about ½" deep. No evidence of cracking could be found except very near the surface.

A pile which had been very badly cracked and then stored in the salt atmosphere for two months was carefully cut in two at the crack. The spirals and strand showed no evidence of corrosion or distortion. Another pile on which driving had been continued to complete failure was reported to have had the spiral broken and the strands twisted.

Recalling the papers presented by Henry J. Cowan of Australia on torsion, and his statement that torsion cracks did not close, the evidence was beginning to point towards torsion, even though the cracks were horizontal instead of inclined at 45 degrees. The question was how the torsion might have been introduced.

On all broken piles and some unbroken ones, it was noticed that the driving head walls had chiseled a long scar or scratch on 4 faces, adjacent to the corner. Most unbroken piles had the scar on one face only, generally on the entire face.

Then after six days and seventy piles of successful driving, the word came that another pile had cracked. This pile was 70 feet long, being driven in a location originally scheduled for a 55 foot pile, which proved fortunate for the investigation.

The crack had appeared 18 feet down from the head, and was puffing concrete dust at every blow. Driving was stopped. The crack did not close.

What had happened was that there was a new foreman on the piledriver that morning and he had not filled the driving head, so the driving head was socketed about 9" over the head of the pile. The pile had rotated about 25 degrees so that it was jammed against the walls of the driving head on all four faces. The walls had chiseled down the four faces adjacent to the corner. Every blow of the hammer was imparting an impact twist to the pile.

The hammer and the driving head was raised. The stern of the floating driver was swung around about 25 degrees until the driver and the driving head was aligned with the pile. Then the driving head was lowered back onto the pile and driving was recommenced. The pile was then successfully driven to design tip elevation, the last foot requiring 200 blows. Not only did no further cracking occur, but the original crack no longer "puffed" any concrete dust nor spalled any flakes of concrete.

Since this particular pile was fortunately much longer than called for in this location, the original crack was above water and visible during the entire driving operation.

Following this occurrence, the procedure was adopted on all remaining piles of making the top 12" of the pile round in cross-section so it can rotate without restraint in the driving head. The driving head has also been filled to reduce the length of socket.

Thus an 18" octagonal pile is made 18" round at top, a 16" octagonal is made 16" round, and a 14" square pile is made 14" round. On piles already cast, the head was carefully chipped to round by hand, using a small hammer.

About 2000 piles have subsequently been driven without a single occurrence of torsional cracking.

The second project in which torsion cracks appeared involved the driving of 21" octagonal prestressed piles with a 12" hollow core, but solid head and tip. This was a wharf project in North Charleston, South Carolina.

The piles were being driven through water into a sloping bank of hard clay. Serious cracking occurred, with the initial crack about 15' below the head, on 60 foot long piles. Most of these cracks were underwater when they initially opened and thus were not noticed until the pile suddenly failed completely.

In one case, the steel leads, which were rather light, collapsed in twisting.

At that time torsion was not suspected and the problem was believed to lie with the hollow cores. So a change was made to solid piles 21" octagonal, with a consequent increase in the total concrete area and prestress force. By this step, cracking failures were kept to an acceptable minimum. It is my opinion that the added torsional resistance of the solid pile prevented or reduced torsional cracking.

About this same time, Mr. Coleman Smith, Resident Engineer on this project for Lockwood-Greene and Associates, Con-

sulting Engineers, noticed that all piles, broken and unbroken, twisted during driving and tended to bind in the driving head. He therefore had the corners at the head of the pile chipped off to make the head round. But since several changes had been made simultaneously, and since the driving had by this time progressed to a softer and easier area, there was no way to determine the relative importance of the factors involved.

Recent investigation of the broken piles on this project showed that the heads had been severely scarred or chiseled by the driving head on 4 faces adjacent to the corners and that final failure did occur on a 45 degree inclined plane.

The sudden collapse of a pile during driving has occurred on several isolated occasions on other projects. Could these have also been torsion failures occuring under water or underground, with the initially cracking passing unnoticed?

The third project was a test pile for a wharf in Portland, Oregon, involving 120 foot long prestressed piles, 20" square, with a 12" hollow core, solid head and tip. Because only a few piles were involved, the pile was set by a derrick and then the hammer held in a driving frame or yoke, was set over the head of the pile. The way in which the hammer was rigged caused it to swing naturally at a 45 degree orientation to the sides of the pile. The driving frame, which extended 10 feet down on the pile had a square steel guide that fitted closely around the pile.

During comparatively easy driving these piles developed horizontal cracks about 30' down from the head. Under continued driving, more cracks appeared at about 2 foot increments above and below the initial crack, with final failure being a beautiful 45 degree helical surface. The puffing of dust and surface spalling were typical. Heads were scarred on two faces near the corners. The cracks looked very bad, due to the spalling, but were only about ½" deep.

The pile head was then chipped to round and the guide frame was also made round. The remaining 12 test piles were then successfully driven to the required bearing. However, close examination of two of these 12 piles which had been driven with round heads showed small cracks about 35' down from the head. These were incipient cracks of the same kind that had been previously noted, and on very close examination, could be traced completely around the pile. At two corners some spalling had occurred. An examination of the heads of these two particular piles showed that the round heads had been chipped and spalled to a rough polygon and had then been subject to the same impact twisting as before. Therefore, it would seem essential that not only should the pile heads be made round but the driving helmet should not overlap the head of the pile very far and every effort should be made to so rig the hammer as to prevent or minimize a tendency to twist.

These three incidents of torsion in the driving of prestressed concrete piles recalls similar phenomena in driving conventionally reinforced precast concrete piles. There have been several cases where severe pile cracking and spalling has occurred on conventionally reinforced piles approximately twenty feet or more below the head of the pile with little or no apparent damage to the head of the pile. These cracks were underwater and were not discovered until a large number had been driven. One of these cases was a large naval pier at Alameda, California, where 18" square piles were jetted and driven into a dense sandy clay. The other instance was a pier at the Port of Long Beach, California, where the piles were driven through 15 feet or so of 3" rock which had been previously placed on top of reasonably firm sand. These two cases of cracking were assumed at the time to be due to bending and to hard driving. The nature of the cracking and spalling indicates that torsion may have been the true primary cause.

It has been reported that in driving conventionally reinforced precast concrete piles at a naval installation at Subic Bay, Luzon, Philippine Islands, the piles twisted as much as 90 degrees or more during driving. The piles transmitted this torque through binding in the driving head to the leads, which were thereby twisted and damaged. A new driving head was installed which rotated freely and this eliminated damage to the leads. No damage to the piles was noted on the job.

A very interesting experimental program was performed in 1956 by the U. S. Naval

Research and Evaluation Laboratory of Port Hueneme, and the Port of Long Beach, California. (Engineering News Record, August 9, 1956, "Prestressed Piles Will Take More Punishment".) Precast concrete piles, 18" square x 80 feet long, were well instrumented with strain gauges and then driven with continuous diver inspection for cracks. These showed considerable cracking and spalling after driving resistances of 10 blows per inch with a Vulcan 80-C hammer. Cracks initially occurred about 30 feet from the head, at a time when the penetration was 28 feet. These later spread, but in general were concentrated about midway of the pile length above the ground line. Initial hairline cracks became 1/16" wide under repeated blows followed by spalling. The maximum tensile strains recorded were all several times the modulus of rupture, and corresponded to a tensile stress in the range of 1000 to 3400 psi. If these cracks had been due to bending, then the largest cracks should have been on one face only and at or near the ground line. But the cracking and spalling was about midway between the ground and pile head and was on all four faces.

14" octagonal prestressed piles were test driven under this same program. It was noted at the time that the pile head fit loosely in the driving head and it was feared this might cause spalling. However, these piles suffered no apparent damage whatsoever despite driving resistances as high as 200 blows to the ½ inch.

The damage to the conventionally reinforced piles was generally assigned to hard driving. It now appears possible, though certainly not conclusive, that torsion may have been a prime factor. The only published data on torsion of prestressed concrete of which this writer has knowledge, are:

1. "Experiments on the Torsional Strength of Prestressed Concrete," Henry J. Cowan, Australia, Proceedings of the Third Congress of the Federation Internationale de la Precontrainte, 1958.
2. "The Torsional Strength of Prestressed Concrete," Henry J. Cowan and Steward Armstrong, Proceedings, World Conference on Prestressed Concrete, San Francisco, California, 1957.

As stated earlier, this report from several isolated field cases is purposely made prior to the obtaining of necessary research and test data that will be necessary to arrive at any definite conclusions. It is hoped that it may serve to prevent unnecessary damage to precast concrete piles, whether prestressed or conventionally reinforced, which may be driven in the near future, by indicating a probable cause and suggesting procedures which will eliminate or minimize the possibility of damage during driving.

This procedure which has been successfully adopted and proven, is intended to permit the pile to twist freely without restraint during driving, and particularly without receiving an impact torsion blow.

In some of the cases investigated, deficiencies in pile manufacture and driving techniques tended to mask the real cause. Without condoning any reduction in standards, it now appears that minor deficiencies will not in themselves cause a sudden or complete failure of the type described as long as torsion is eliminated.

There are several questions yet to be answered, although hypotheses can of course be formed. Some of these are:

1. Why is the initial crack horizontal instead of inclined?
2. Why is its locations so relatively constant at approximately one-third of the distance from the pile head? Is this location determined by the tensile rebound stresses?
3. What is the exact nature of the stresses and the cause? Is it a combination of tensile rebound stresses and torsion stresses as appears probable, and if so, what is the relative influence of each? Is buckling involved in any way?
4. What is the influence, if any, of the extremely high instantaneous compressive stresses (up to 12000 psi) which have been reported to occur under the hammer blow, on the direction and character of the crack?

Torsion, both alone, and in combination with axial stress, would seem to be a fruitful field for greater study and research. Meanwhile it appears that proper driving procedures will prevent damage from torsion during driving and permit the completely satisfactory installation of precast concrete piles.

Tests on 15-year-old, 54-ft-long, Pre-tensioned Beams

By G. D. Base, B.Sc., and H. E. Lewis, M.Sc., D.I.C.

Reprinted from THE CONSULTING ENGINEER—February/March, 1958

Tests on 15-year-old, 54-ft-long, Pre-tensioned Beams

By G. D. Base, B.Sc., and H. E. Lewis, M.Sc., D.I.C.

TWO pre-tensioned *I*-beams, 54 ft long, produced by the long-line method in 1940 for the Ministry of Transport, have been tested by the Cement and Concrete Association at the British Railways Concrete Depot, Newton Heath, Manchester. The main object of the tests was to determine the loss of prestress that had occurred in 15 years, but the tensile strength of the concrete and the ultimate loads and deflections of the beams were also measured. Strain readings, measurements of central deflections, and visual observation, enabled the prestress remaining in the beams to be determined from the loads at which previously formed cracks re-opened.

The beams had been stored unloaded during the whole of their life and the initial prestress in the bottom fibres of the concrete was 1,744 lb/in.^{2}. At the time of testing this had decreased to 1,443 lb/in.^{2}, a loss of 17 per cent. The figure of 20 per cent. for the loss of prestress in a pre-tensioned beam has been adopted by many engineers in the absence of corroboration by experiment. Recently the correctness of this figure has been questioned and the tests described in this report are among a number made or being made to obtain a more accurate figure.

The two beams tested were of the same design and were from a number of beams made for the Ministry of Transport at the outbreak of war for emergency bridging. The majority of these beams have been used but the two tested had been stored without loading since the time of manufacture. The long-line method was used for their manufacture, the line being approximately 940 ft long and one of the longest ever used. These beams were among the first to be prestressed in this country. Fig. 1 gives details of the beams: the derivation of stresses is given below. For the tests a 53 ft span was used with two-point loading, the loads being placed symmetrically on the beam and 16 ft apart.

Cross-section area = 217 in.^{2}
I_c = 19,770 in.^{4}

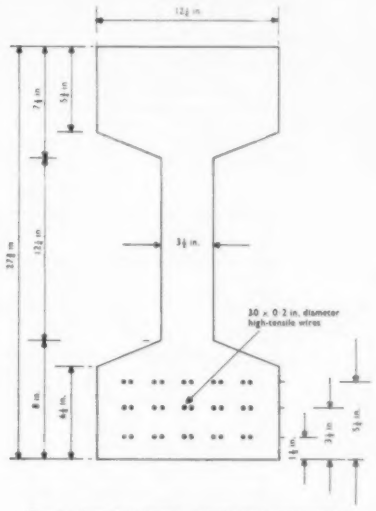


Fig. 1.—Detail of beams. Length 54 ft, supported at 53 ft centres

TABLE 1.—Calculation of I_c for Section

Section	I_1	A	h	Ah^2	$I + Ah^2$
12·25 / 27·75	$\frac{12.25 \times 27.75^3}{12} = 21,814.4$	339.9	13.875 — 13.74 = 0.14	6.7	+ 21,821.1
8·75 / 12·25	$\frac{8.75 \times 12.25^3}{12} = 1,340.4$	8.75 × 12.25 = 107.19	14.125 — 13.74 = 0.39	16.3	— 1,356.7
1·75 / 8·75	$\frac{8.75 \times 1.75^3}{36} = 1.3$	½(8.75 × 1.75) = 7.66	20.25 + 0.58 — 13.74 = 7.09	384.8	386.1
1·75 / 8·75	$\frac{8.75 \times 1.75^3}{36} = 1.3$	7.66	13.74 — (6.25 + 1.16) = 6.33	306.0	— 307.3
					19,771.2

I_c = 19,770 in^4 to four significant figures

Centroid = 13.74 in. above bottom of section

Stresses due to self weight—

Weight per ft :

$$= 217 \times \frac{150}{144} \text{ lb}$$

Central moment :

$$= 217 \times \frac{150}{144} \times \frac{53^2}{8} = 79{,}600 \text{ lb/ft}$$

Stress in bottom fibres :

$$= -\frac{79{,}600 \times 12 \times 13.74}{19{,}770} = -664 \text{ lb/in.}^2 \text{ (tension)}$$

Stress in top fibres :

$$= \frac{79{,}600 \times 12 \times 14.01}{19{,}770} = +678 \text{ lb/in.}^2 \text{ (compression)}$$

Stresses due to prestress—

Wire area :

$$= 0.944 \text{ in.}^2$$

Initial stress in wire :

$$= 70 \text{ tons/in.}^2$$

Initial force in wire :

$$= 70 \times 0.944 = 66.1 \text{ tons}$$

Eccentricity :

$$= 13.74 - 3.42 = 10.32 \text{ in.}$$

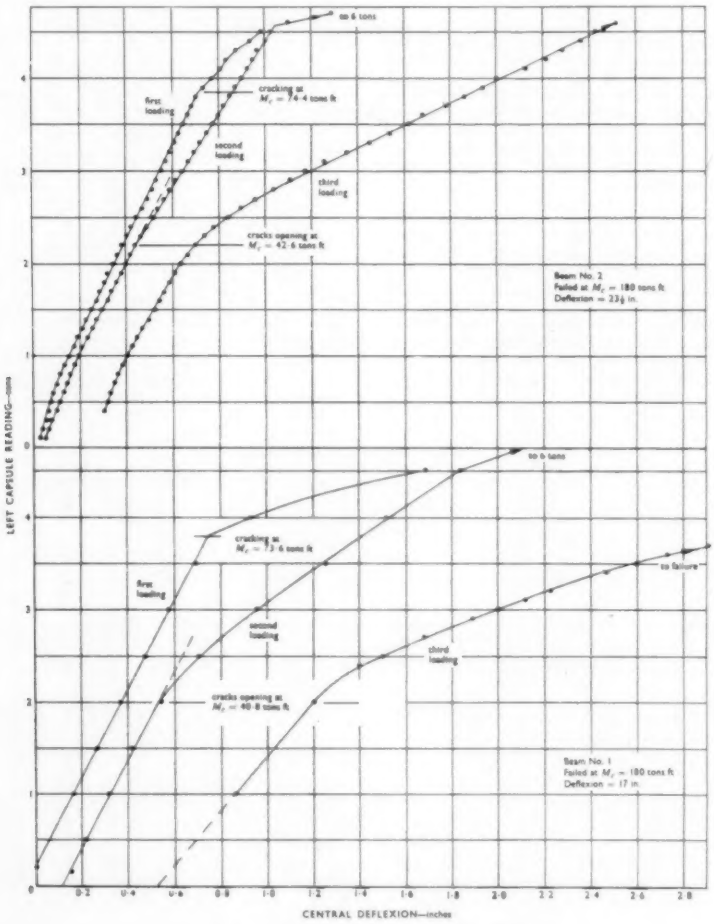

up to 6 tons
first loading
cracking at M_c = 74·4 tons ft
second loading
third loading
cracks opening at M_c = 42·6 tons ft
Beam No. 2
Failed at M_c = 180 tons ft
Deflexion = 23½ in.
up to 6 tons
cracking at M_c = 73·6 tons ft
first loading
second loading
third loading
up to failure
cracks opening at M_c = 40·8 tons ft
Beam No. 1
Failed at M_c = 180 tons ft
Deflexion = 17 in.
LEFT CAPSULE READING—ins
CENTRAL DEFLEXION—inches

Fig. 2.—Load-deflection graphs for both beams tested

Prestress in bottom fibres :

$$= \frac{+66.1 \times 2{,}240}{217} + \frac{(+66.1 \times 2{,}240 \times 10.32 \times 13.74)}{19{,}770}$$

$$= 681 + 1{,}063 = 1{,}744 \text{ lb/in.}^2 \text{ (compression)}$$

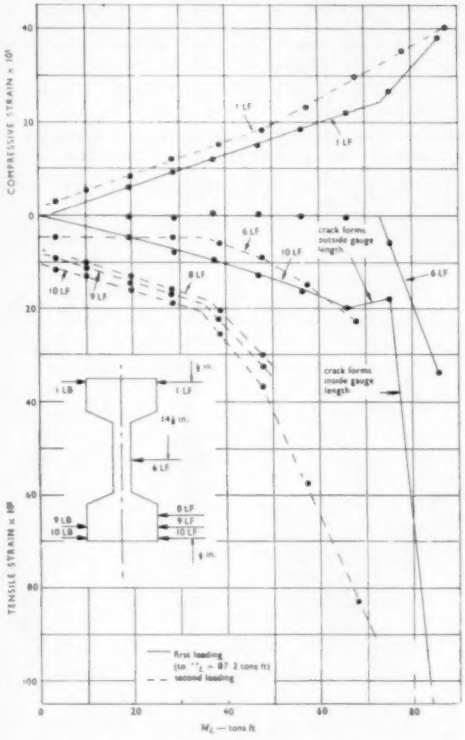


Fig. 3a.—Moment-strain graphs. First beam (left front bank)

Prestress in top fibres :

$$= +681 + \left(\frac{-66.1 \times 2{,}240 \times 10.32 \times 14.01}{19{,}770}\right)$$

$$= +681 - 1{,}084 = -403 \text{ lb/in.}^2 \text{ (tension)}$$

Stresses due to self weight and prestress (53 *ft span*)—

Stress in bottom fibres :

$$= +1{,}744 - 664 = +1{,}080 \text{ lb/in.}^2 \text{ (compression)}$$

Stress in top fibres :

$$= -403 + 678 = +275 \text{ lb/in.}^2 \text{ (compression)}$$

Details of Tests

The two beams were set up in identical rigs and supported on 53 ft spans. A steel roller with distribution plates was used at one support and 2 in. rectangular steel blocks at the other. Loads were applied 8 ft to either side of the centre by a pair of hydraulic jacks fed from a single hand pump through a branching lead. During the early stages of testing, load-measuring capsules above the jacks indicated the load and the pump pressure gauge was calibrated at the same time. When the deflections increased the capsules were removed to produce greater stability of the rig and the pump gauge was used to indicate the load. The travel of the jacks was only 6 in., and repacking was necessary several times before the ultimate deflection of nearly 2 ft was obtained. Central deflections were measured by a 2 in. travel 0.001 in. dial gauge during the early stages of testing and thereafter by a long-travel 0.01 in. vernier gauge. The results are plotted in Fig. 2.

Strains in the concrete were measured with the Demec gauge[1,2] on two banks of gauge lengths up the sides of the beams on either side of the centre, with check readings on the backs of the beams. The results are plotted in Figs. 3 and 4. A small microscope was used to search for cracks. The beams were first loaded until soon after cracking occurred. On the first beam, strains and deflections were read for every ½-ton increment at each load point but near the cracking load particular care was taken to observe the deflection gauge and the central area of the beam. The commencement of cracking was obvious from the sound and from the sudden increase in deflection. The cracks immediately penetrated the whole

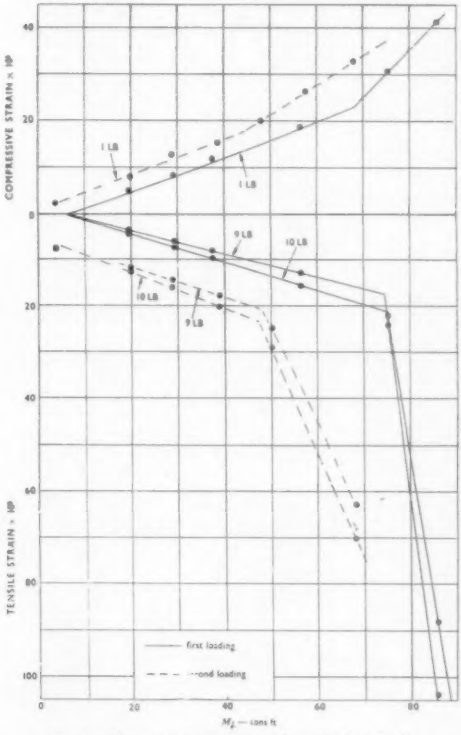


Fig. 3b.—Moment-strain graphs. First beam (left back bank)

of the bottom flange. On the second beam strains were again read for every ½-ton increment but deflections were read for every $^1/_{10}$-ton increment. Cracking was again obvious, but the increase in deflection was not as great as for the first beam.

The loads were removed after the beams had cracked and then re-applied in a similar way so that the loads necessary to re-open

the cracks could be measured. This second loading was continued to 6 tons at each point to extend cracking, and the loads were again removed (though a small load remained applied owing to friction in the jacks). It was noted that the residual deflection of the first beam was considerably greater than that of the second. Loading was then re-applied until collapse occurred. Both beams failed at approximately the same load, the deflection of the first, however, being rather less than that of the second. In both beams the steel was obviously near its breaking-point when the concrete crushed in the top flanges, and in the first beam the steel broke immediately after the concrete had crushed.

Results

The initial cracking loads of the two beams were easily determined from deflection curves (Fig. 2) and from observation. Although the strain readings (Figs. 3 and 4) gave a rough check, in the first beam the first cracks formed outside the area with gauge lengths, causing a relief of strain on the gauge lengths and making graphical use of the readings difficult. In the second beam there was a small discrepancy between gauge banks, as cracks did not form in the two places at exactly the same time. One bank

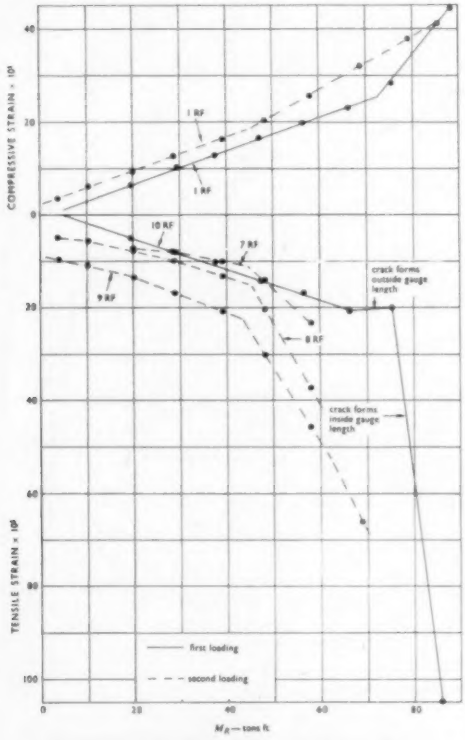


Fig. 3c.—Moment-strain graphs. First beam (right front bank)

(Fig. 4*a*) in fact indicates an initial cracking moment slightly below that given by the deflections, probably owing to a single crack which was insufficient to influence the deflection. (It should be noted that the bending moments plotted as M_L and M_R in Figs. 3 and 4 do not include 1.9 tons/ft due to the loading jacks and capsules.) From the deflection curves and observation the initial cracking moments were :

first beam	73.6 tons/ft
second beam	74.4 tons/ft

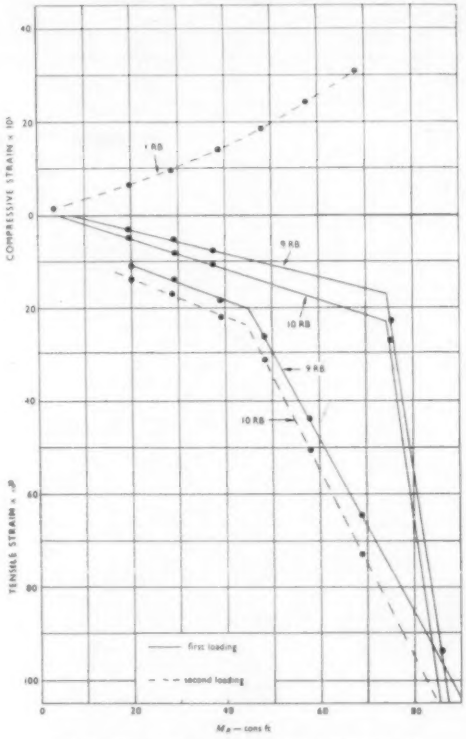


Fig. 3d.—Moment-strain graphs. First beam (right back bank)

The " premature " value for the second beam, obtained from the strains, was 72.5 tons/ft. After cracking, the behaviours of the two beams differed considerably. The deflection of the first beam increased from approximately 0.75 in. to 1.7 in., and that of the second beam from approximately 0.72 in. to 1.0 in. between cracking and 4½ tons. The values of E up to cracking were almost identical, and the crack patterns were similar. On removal of the loads all cracks closed. The residual deflection, however, was somewhat greater for the first than for the second beam.

The re-opening of the cracks in the first beam, as determined from the deflection curve and from observation, was well defined. Strain readings, however, indicate a re-opening at a somewhat lower load (Fig. 3*a*).

This could have been the result of straining the steel across one crack beyond its elastic range, resulting in a local permanent extension and loss of prestress. The value obtained from the deflection curve and by observation was 40.8 tons/ft, although the figure indicated by the left strain gauge bank was 37.5 tons/ft. The re-opening of the cracks in the second beam was not easily visible but the figure of 42.6 tons/ft given by the deflection curve appears quite definite. Again, however, one strain gauge bank indicates a lower figure of 40.5 tons/ft (Fig. 4*a*).

Both beams had ultimate loads that appeared to be the same within the limits of accuracy of the pump gauge and gave a moment of 180 tons/ft. The ultimate deflections were not the same, as the first beam collapsed at a deflection of 17 in. and the second beam at a deflection of 23½ in. Crushing of the upper flange concrete occurred in both beams, but in the first beam the steel broke immediately afterwards. Fig. 5 shows that, before collapse, only about the top 3½ in. of the beam was uncracked. The value of E for the concrete till the time of cracking was almost the same for both beams and was calculated from the expression :

$$\text{central deflection} = \frac{Wa\,(3L^2 - 4a^2)}{48\,EI}$$

For a load W of 2 tons, a distance a between support and loads of 18.5 ft, and a span L of 53 ft, the measured deflection was 0.2 in. Hence :

$$E = 5.33 \times 10^6 \text{ lb/in.}^2$$

For calculation of loss of prestress and tensile strength of the concrete, the initial cracking loads for the two beams and also the loads to re-open the cracks were close enough to allow of averaging.

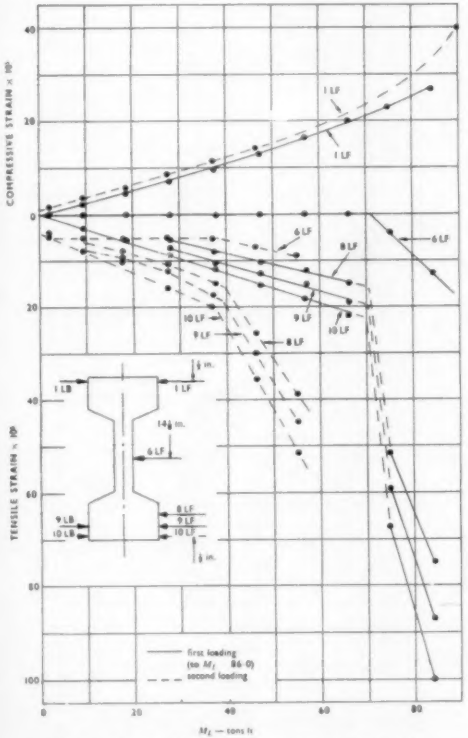


Fig. 4a.—Moment-strain graphs. Second beam (left front bank)

Then, average applied moment at initial cracking (from deflections) :

$$= \frac{73.6 + 74.4}{2} = 74.0 \text{ tons/ft}$$

Average applied moment at crack re-opening (from deflections) :

$$= \frac{40.8 + 42.6}{2} = 41.7 \text{ tons/ft}$$

Hence, increase in stress of the bottom fibres to produce re-opening of previously formed cracks is :

$$\frac{41.7 \times 2{,}240 \times 12 \times 13.74}{19{,}770} = 779 \text{ lb/in.}^2$$

and the tensile strength of the concrete is :

$$\frac{(74 - 41.7) \times 2{,}240 \times 12 \times 13.74}{19{,}770} = 604 \text{ lb/in.}^2$$

Thus loss of prestress in bottom fibres = 1,080 — 779 = 301 lb/in.²

$$\text{Percentage loss} = \frac{301 \times 100}{1{,}744} = 17$$

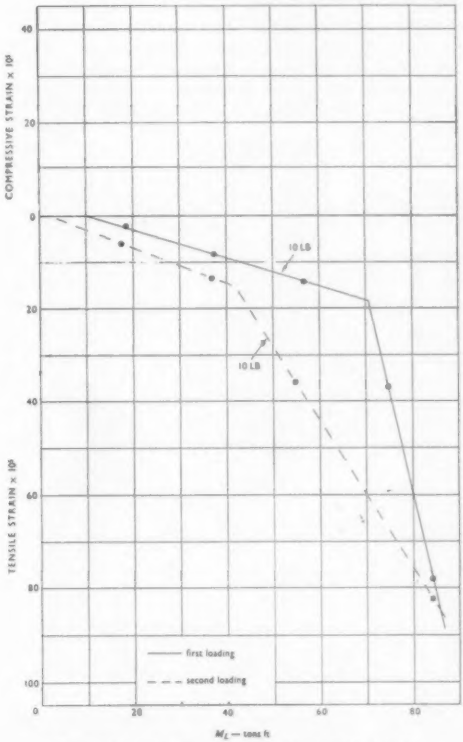


Fig. 4b.—Moment-strain graphs. Second beam (left back bank)

Tensile tests on a few samples of the wire taken from the ends of each beam gave almost identical results for the two beams. The initial modulus of elasticity of the wires was 25.5×10^6 lb/in.² and the ultimate load was 3.19 tons. The extension of a 10 in. gauge length which included the rupture was approximately 0.15 in.

The theoretical ultimate bending moment for the beams, assuming steel failure and a depth of concrete under compression of $3\frac{1}{2}$ in., is 180 tons/ft and agrees exactly with the observed figures.

Theoretical Calculation of Loss of Prestress

The loss of prestress will depend upon a number of variables, and a method of forecasting losses must be devised. The following method is from the draft of the Code of Practice for Prestressed Concrete and overestimates the loss of prestress.

Assume :

Initial stress in bottom fibres of concrete	= 1,080 lb/in.^{2}
and stress at steel level	= 980 lb/in.^{2}
Initial wire stress = 70 tons/in.^{2}	= 156,800 lb/in.^{2}
and modulus	= 28×10^6 lb/in.^{2}

A minimum cube strength of 3,150 lb/in.^{2} at transfer was specified but records show that cube strengths of as much as 6,250 lb/in.^{2} were obtained. It was not possible to decide the cube strength of the actual beams tested and the theoretical losses are thus calculated here, assuming (1) a cube strength of 3,200 lb/in.^{2} and a steel relaxation of 15,000 lb/in.^{2}, and (2) a cube strength of 6,300 lb/in.^{2} and a steel relaxation of 10,000 lb/in.^{2}.

(1) *Assuming a cube strength of* 3,200 *lb/in.^{2} at transfer—*

(*a*) Loss due to steel relaxation = 15,000 lb/in.^{2}. (This could be zero for some modern steels.)

(*b*) Loss due to elastic deformation of concrete (assuming $E = 3.2 \times 10^6$ lb/in.^{2} for 3,200 lb/in.^{2} concrete) :

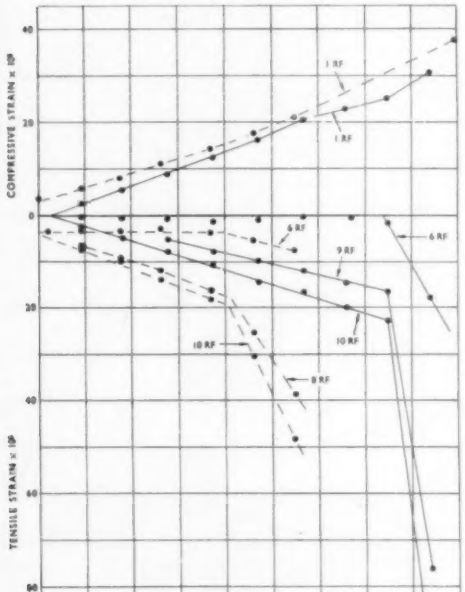


Fig. 4c.—Moment-strain graphs. Second beam (right front bank)

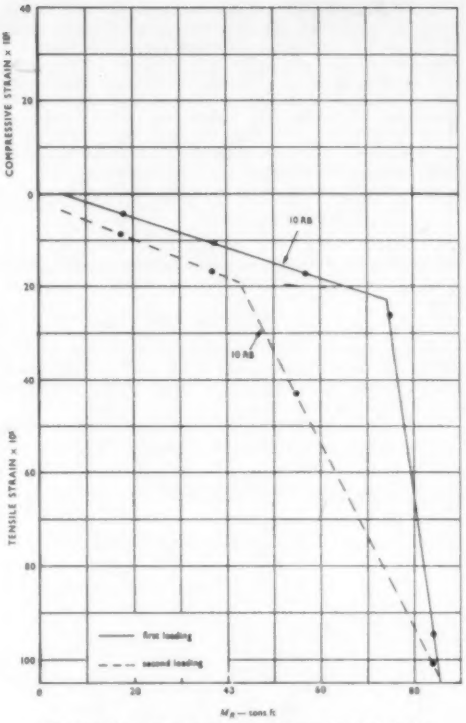


Fig. 4d.—Moment-strain graphs. Second beam (right back bank)

(*c*) Loss due to shrinkage of concrete :

$= (300 \times 10^{-6}) \times (28 \times 10^6) = 8{,}400$ lb/in.^{2}

(where 300×10^{-6} is the ultimate shrinkage of the concrete)

$$= \frac{28 \times 10^6 \times 980}{3.2 \times 10^6} = 8{,}590 \text{ lb/in.}^2$$

(*d*) Loss due to creep of concrete :

$$= 0.33 \times 10^6 \times \frac{6{,}000}{3{,}200} \times 980 \times 28 \times 10^6$$

$= 17{,}000$ lb/in.^{2}

where 0.33×10^{-6} and 6,000 are constants.

Total loss = 50,800 lb/in.^{2}

$$\text{Percentage loss} = \frac{48{,}990}{156{,}800} \times 100 = 31$$

(2) *Assuming cube strength of* 6,300 *lb/in.^{2} at transfer—*

(*a*) Loss due to steel relaxation = 10,000 lb/in.^{2}

(*b*) Loss due to elastic deformation of concrete (assuming $E = 5.15 \times 10^6$ lb/in.^{2} for 6,300 lb/in.^{2} concrete)

$$= \frac{28 \times 10^6}{5.15 \times 10^6} \times 980 = 5{,}300 \text{ lb/in.}^2$$

(*c*) Loss due to shrinkage of concrete = 8,400 lb/in.^{2}

(*d*) Loss due to creep of concrete

$$= 0.33 \times 10^{-6} \times \frac{6{,}000}{6{,}000} \times 980 \times 28 \times 10^{6}$$

$$= 9{,}050 \text{ lb/in.}^2$$

Total loss $= 32{,}750$ lb/in.^{2}

$$\text{Percentage loss} = \frac{32{,}750}{156{,}800} \times 100 = 20.9$$

Conclusions

The two tests carried out by the Cement and Concrete Association gave similar results, which can therefore be averaged. Bending moments at which cracks formed and re-opened appeared to be clearly defined by load-deflection curves and these values have been used to determine the loss of prestress. Strain readings showed earlier local re-opening, but as this might have been due to local yielding of the steel these figures are not being considered until further tests can be carried out under better conditions. These early cracks were not discovered by visual observation and were not sufficient to affect the general behaviour of the beams. The loss of prestress deduced from deflections was apparently 17 per cent. The tensile strength of the concrete was 600 lb/in.^{2}.

No explanation can be found for the more rapid increase in deflection after initial cracking of the first beam compared with the second. If this increase was owing to greater bond slip the steel of the first beam should not have broken when the beam failed. A difference in the steels would account for the phenomenon, but it seems probable that both beams were from the same line and results of tests on samples of the steel taken from the ends of each beam support this inference. A theoretical basis on which the loss of prestress could be forecast is given. Neither the cube strength of the concrete at transfer, nor the relaxation of the steel, is accurately known and the value obtained assuming the specified minimum cube strength and the maximum steel relaxation loss is considerably greater than the real value. Assuming a higher cube strength and a smaller loss due to steel relaxation gives a closer agreement although the figure is still slightly greater than that obtained experimentally. Use of the actual modulus of elasticity of the steel instead of that recommended in the draft Code of Practice makes the agreement still closer.

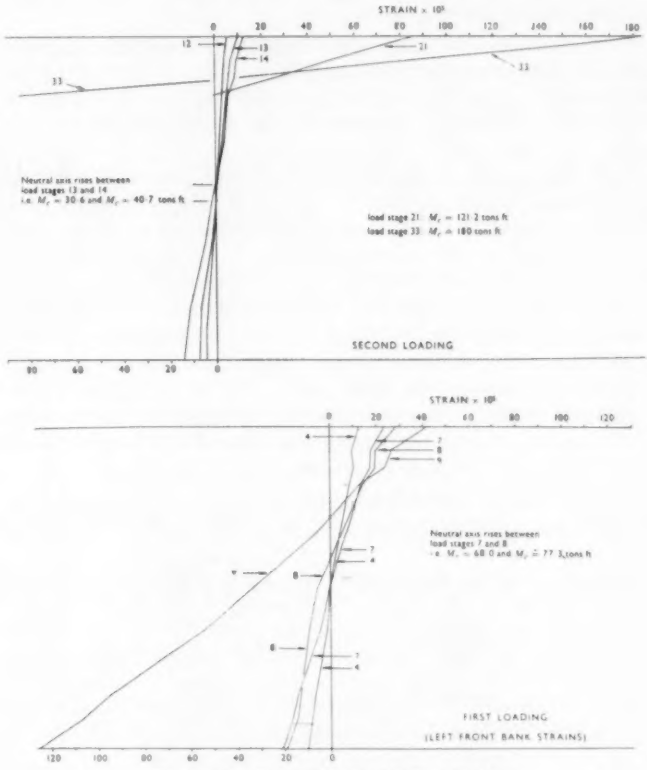

STRAIN × 10⁶
0 20 40 60 80 100 120 140 160 180
12
13
14
21
33
33
Neutral axis rises between load stages 13 and 14 i.e. M_r = 30·6 and M_r = 40·7 tons ft
load stage 21: M_r = 121·2 tons ft
load stage 33: M_r = 180 tons ft
SECOND LOADING
80 60 40 20 0
STRAIN × 10⁶
0 20 40 60 80 100 120
4
7
8
9
Neutral axis rises between load stages 7 and 8 i.e. M_r = 68·0 and M_r = 77·3 tons ft
7
8
8
7
4
FIRST LOADING
(LEFT FRONT BANK STRAINS)
120 100 80 60 40 20 0

Fig. 5a *and* b.—Strain profiles due to loading. First beam

Two similar beams are shortly to be tested under laboratory conditions, and the testing of other units with greater initial prestress is also under consideration.

References

[1]MORICE, P. B., and BASE, G. D. "The design and use of a demountable mechanical strain gauge for concrete structures." *Magazine of Concrete Research*. Vol. 5, No. 13. August, 1953, pp. 37-42.

[2]BASE, G. D. "Further notes on the Demec, a demountable mechanical strain gauge for concrete structures." *Magazine of Concrete Research*. Vol. 7, No. 19. March, 1955, pp. 35-38.

BUILDING CODE REQUIREMENTS FOR PRESTRESSED CONCRETE

SUB-COMMITTEE ON PRESTRESSED CONCRETE
STRUCTURAL ENGINEERS ASSOCIATION OF NORTHERN CALIFORNIA

The rapidly increasing use of prestressed concrete in this country calls for the immediate incorporation of its specifications in our building codes. Such inclusion will help building officials in checking designs using prestressed concrete. It will also serve as a basis for design and construction engineers.

With the above in mind, the Structural Engineers Association of Northern California established a subcommittee on prestressed concrete. This subcommittee has prepared a draft for the inclusion of prestressed concrete in the Uniform Building Code. This draft has been submitted to the Board of Directors of the Association for approval and further action.

The Uniform Building Code, by the Pacific Coast Building Officials' Conference, is the prevalent official document widely adopted throughout the Western States. Chapter 26 of this Code entitled "Concrete" presently contains regulations pertaining to plain, reinforced, and precast concrete. It is intended to add prestressed concrete as an extension of this chapter.

This draft is essentially based on the material contained in the "Tentative Recommendations for Prestressed Concrete" by ACI-ASCE Joint Committee 323, 1958, and the specifications published by the Prestressed Concrete Institute. In order to conform to the Uniform Building Code in language and style, a considerable amount of work was required in the process of writing this draft.

Since the Uniform Building Code contains all materials and methods of construction, it was necessary to write the section for prestressed concrete in a rather condensed version. Certain deviations from the ACI-ASCE tentative recommendations were made at the committee's discretion. Great care was taken that the various clauses will serve as guidance and criteria but will not tend to restrict the development of this young and growing industry.

The subcommittee consisted of Theodore C. York, Chairman; F. W. Cheesebrough; R. C. Clark; G. F. Durbin; F. D. Gaus; D. Higgins; L. A. Irvin; W. F. Koller; Prof. T Y. Lin; R. A. McCann; M. Silvert and Dr. G. A. Tarics.

PROPOSED CHANGES TO UNIFORM BUILDING CODE CHAPTER 26 INCORPORATING PRESTRESSED CONCRETE

Sec. 2601. The following additions to be made:

MATERIALS	U.B.C.	ASTM
Specifications for air-entraining Portland Cement	———	C175
Portland Blast Furnace Slag Cement	———	C205
Portland Pozzolan Cement	———	C340
Prestressing Tendons:		
Strand for Pretensioned Concrete	———	A416-57T
Uncoated stress relieved wire for Prestressed Concrete	———	A——58T
Alloy Steel Bars	———	
Strand for Post-tensioning	———	

U.B.C. Designations to be established for above items.

Sec. 2602. The design of structures in concrete of cast-in-place or precast construction, plain, reinforced or prestressed, shall conform to the rules and principles specified in this chapter.

Sec. 2603. Add Definitions:

Cast in place construction, construction whereby concrete is deposited in its final position.

Precast construction, construction whereby concrete members are cast in a place prepared for this purpose, not necessarily on the building site, to be moved to their final position - - - -.

Prestressed concrete construction, construction of concrete members, cast in place or precast, in which controlled forces create predetermined internal forces and moments other than those which are caused by the application of further Dead, Live and other external loads.

Prestressing force is the force which is designed and controlled to create the required internal forces and moments.

Tendons are steel members which generally are used to exert the prestressing force onto the concrete.

Transfer is the act of application of the prestressing force to the concrete.

Pretensioning is the system of prestressing concrete members whereby tendons are stressed prior to pouring of the concrete and the prestressing force is transmitted to the concrete by bond.

Post-tensioning is the system of prestressing concrete members whereby tendons are stressed after the concrete has obtained a required strength and the prestressing force is transmitted to the concrete generally by anchoring the stressed tendons to the member.

Losses:

a) **Gradual Losses** are the reduction from the initial force to the working force and are due to shrinkage, creep of the concrete and relaxation of the steel.

b) **Elastic Shortening** losses are due to the elastic deformation of the concrete at the time of transfer.

c) **Tensioning losses** are those incurred in post-tensioned concrete at the time of stressing and anchoring the tendons, they are due to tendon friction and anchorage slip and may vary with the system used.

Forces:

a) **Jacking Force** is the force exerted by the ram on the tendon.

b) **Initial Force** is the force left in the tendon immediately after transferring the prestressing force to the concrete.

c) **Working Force** is the final force left in the tendons after all losses have occurred.

Sec. 2604.

c) **Add:** Seawater shall not be used for the construction of prestressed concrete members.

e) Prestressing Steel shall conform to the requirements set forth in U.B.C. Standards No.

f) Previously was e).

g) Caution shall be used that no contact exists between stressed steel and concrete in which an admixture has been used that contains Calcium Chloride.

Sec. 2606.

Under table **26-A** add note:

Concrete of higher strengths shall be designed by a qualified person or a laboratory.

Sec. 2609.

f) - - - - - - - - - and three days for high-early-strength cement, which period may be reduced if high temperature curing is applied. High temperature curing shall conform to latest practice. The rate of heating and cooling shall be controlled so as to prevent thermal shock to the concrete. During high-temperature curing the concrete shall be protected from rain and the rapid loss of moisture prior to the curing period. Curing shall be continued until the required strength for transfer is reached. Rapid drying should be prevented, where it would tend to produce undesirable cracks.

Sec. 2610.

a) add: Forms for prestressed concrete members shall be designed and constructed to permit movement and deflection which take place when the prestressing force is transferred to the concrete.

Sec. 2627.

The design of prestressed concrete members shall be based on both the elastic and ultimate theory. The elastic theory shall be used to determine their performance under design loads. The ultimate strength theory shall be applied to ensure that ultimate capacity provides for the load factors specified in Section 2629.

Sec. 2628. Modulus of Elasticity.

a) Modulus of Elasticity of normal weight concrete may be assumed to be 1,800,000 psi plus 500 times the cylinder strength at the age considered and this value shall be used for the calculation of deflections. When a more exact value is required, as would be required for determining other elastic deformations, it shall be determined by test.

b) Modulus of Elasticity of light weight concrete shall be determined by test.

c) For purpose of determining modular ratios, the Modulus of Elasticity of prestressing tendons can generally be assumed to be:

29,000,000 psi for cold drawn wire.
27,000,000 psi for 7 wire strand.
25,000,000 psi for strand with more than 7 wires.
27,000,000 psi for Alloy Bars.

These values shall however not be used for determining the elongation of tendons, except in preliminary calculations. Elongation of tendons under prestress shall be determined from the stress-strain or load-elongation curves for the lot of steel used.

Sec. 2629. Ultimate Strength theory shall be used to ensure that structures are of adequate capacity for the following load requirements:

. 1.2D + 2.4L
or 1.8 (D+L)
or 1.4 (D + L + W)

Ultimate strength shall preferably be governed by elongation of the prestressing steel rather than by shear, bond or concrete compressing.

Sec. 2630. Allowable stresses.

a) Prestressing steel.
1) Stress in the tendons due to the Working Force after all losses have occurred shall not exceed 60% of the guaranteed ultimate strength of the tendon.
2) Stress in the tendons due to the Jacking Force before transfer shall not exceed 80% of the guaranteed ultimate strength of the tendon.
3) Should, in the case of post-tensioning, friction losses be encountered which are considerably in excess of those assumed by theoretical calculations, so that a Jacking Force of 80% of the guaranteed ultimate strength of the tendon will not provide the required initial or working force, the jacking force may be increased to 85% of the guaranteed ultimate strength, if approval for this increase has been obtained from the Engineer. A stress in excess of 85% of the guaranteed ultimate strength of the tendon shall not be permitted.

b) Stress in concrete immediately after transfer of the prestress shall not exceed the following (f'ci is concrete strength at time of transfer):
1) Compression 0.60 f'ci
2) Tension for members without non-prestressed reinforcement:

Single element	$3\sqrt{f'_{ci}}$
Segmental element	zero

Tension for members with non-prestressed reinforcement provided to resist the tensile force in the concrete, computed on the basis of an uncracked section:

Single element	$6\sqrt{f'_{ci}}$
Segmental element	$3\sqrt{f'_{ci}}$

c) Stresses at design loads after all losses:

1) Compression .45 f'_c

2) Tension

Single element, pretensioned, not exposed to weather	$6\sqrt{f'_c}$
Single element, post-tensioned, not exposed to weather	$3\sqrt{f'_c}$
Segmental element	zero

"These allowable stresses may be exceeded, provided it is shown either by tests or by conclusive analysis that the structure will behave properly under service conditions and will meet all requirements for possible overloads."

d) The bearing pressure created by anchorage in end blocks, for the case of post-tensioned prestressed concrete, shall not exceed 0.6 $f'_{ci}\sqrt[3]{A_c A_b}$ where A_b is the bearing area of the anchor plate and A_c is the maximum area of portion of the member that is geometrically similar to and concentric with the area of the bearing plate, nor shall it exceed f'_{ci}, whichever is the smaller.

Sec. 2631. Losses of Prestress.

a) Losses in prestress from the initial force to the working force are due to elastic shortening, shrinkage, creep of concrete and relaxation of steel, and may be determined from reliable test data and conclusive analysis. In the absence of such data and analysis, the sum of these losses is generally assumed to be

35,000 psi for pretensioned concrete

25,000 psi for post-tensioned concrete.

b) Tensioning losses from the jacking force to the initial force, for the case of post-tensioned concrete, are due to intended and accidental friction, and, for theoretical calculations, may be determined by the following formula:

$T_o = T_x(1 + KL + u\alpha)$, where T_o is the Jacking force T_x is the initial force at the required point.

L is the length in feet from Jack to the required point and α is total angular change from Jack to the required point, expressed in radians.

In the absence of test data, K and u values may be taken from table 26-E.

Table 26-E

Type of steel	Type of duct or sheath	Design Values K	Design Values u
Wire cables	Bright metal sheathing	0.0020	0.30
	Galvanized metal sheathing	0.0015	0.25
	Greased or asphalt-coated and wrapped	0.0020	0.30
High strength bars	Bright metal sheathing	0.0003	0.20
	Galvanized metal sheathing	0.0002	0.15
Galvanized strand	Bright metal sheathing	0.0015	0.25
	Galvanized metal sheathing	0.0010	0.20

c) Actual losses due to friction and wobble shall be verified at the outset of the prestressing operation. Should friction losses be encountered which are considerably in excess of those obtained by calculations specified in b) of this section, the jacking force shall be altered as outlined in Section 2630. a), 3).

Sec. 2632. Shear

a) **General.**—It is essential that shear failure should not occur before ultimate flexural strength is developed. If required, shear capacity can be increased by one of the following methods:

adjusting prestress force
increasing web thickness
increasing web reinforcement

b) **Inclined Cracking.** The formation of inclined cracks should be limited by investigating the principal tension and limiting its maximum value to $0.03f'_c$ under design loads, or $0.08f'_c$ under ultimate loads.

c) **Web Reinforcement.** When principal tension exceeds $0.03f'_c$ under design loads or $0.08f'c$ under ultimate loads, web reinforcement, in the absence of more exact test data, is usually computed from the following formula:

$$A_v = \tfrac{1}{2} \frac{(V_u - V_c)s}{f'_y jd}$$

where A_v = area of web reinforcement at spacing s, placed perpendicular to the axis of the member
V_u = shear due to specified ultimate load and effect of prestressing
$V_c = .06\ f'_c b'jd$, but not more than $180\ b'jd$
b' = thickness of web,
s = longitudinal spacing of web reinforcement
f'_y = yield strength of web reinforcement

Where the web reinforcement is designed by the above, its minimum amount should be $A_v = 0.0025\ b's$.

d) The spacing of web reinforcement shall not exceed three-quarters the depth of the member, unless test-data show a different requirement.

e) Members shall be checked for shear at all critical points. For simply supported members carrying only uniformly distributed load, the critical point shall be assumed to be located at a distance from the support equal to the depth of the member. In the middle third of the span length, the amount of web reinforcement provided should not be less than that required at third-points of the span.

f) At sections of combined high moment and shear, special investigations should be made.

g) For members having span to depth ratios greater than 15, or for uniformly loaded members, web reinforcement may not be required. The omission of web reinforcement may be allowed when justified by test.

Sec. 2633. Ultimate Flexural strength.

Ultimate flexural strength of prestressed concrete may be determined by the following:

$$M_u = A_s f_{su} jd$$

where A_s = area of prestressed or reinforced steel
f_{su} = the ultimate strength of prestressed steel or the yield point of mild steel, to be properly modified if the beam is unbonded or over-reinforced.
jd = the distance between the center of tensile force and the center of compression, to be determined by acceptable methods such as the Whitney or Jensen theories.

Sec. 2634. Construction. Construction of prestressed concrete members shall be in conformity with latest practice. The prestressing operation shall be under direct supervision of personnel fully experienced in this type of work. Prestressing beds for the casting of pretensioned members shall be designed to withstand the forces applied to them.

Sec. 2635. The prestressing tendons shall be accurately located to follow the path outlined in the plans. Care shall be taken that they are not displaced during depositing or vibrating of concrete. Unless tolerances for location of tendons are shown in the plans, variation in location shall not exceed ⅛ inch to ¼ inch, depending on size of member.

Sec. 2636. Prestressing steel shall be kept clean and dry. Except in the case of unbonded post-tensioned construction, foreign matter, grease, oil, paint, shall be removed. A light coat of rust is permissible, provided loose rust has been removed and the surface of the steel is not pitted.

Sec. 2637.

a) **Pretensioned construction** shall conform to all requirements outlined in Sections 2633, 2634 and 2635 of this Code.

b) Force in the tendons shall be determined by measuring elongation and checking jack pressure on a calibrated gage. In case of difference in excess of 5%, the cause of the discrepancy shall be investigated and brought to the attention of the Engineer.

c) Minimum horizontal and vertical clear spacing between pretensioning steel elements at ends of members shall be three times the diameter of the steel or 1-1/3 times the maximum size of coarse aggregate, whichever is greater.

d) Minimum cover over tendons shall be 2½ diameters or one inch, whichever is greater.

e) The force in the prestressing steel shall be transferred to the concrete smoothly and gradually. If the force in the wires or strands is transferred individually, a sequence of release shall be established by the Engineer to avoid subjecting the member to unanticipated eccentricities.

f) Ends of pretensioning steel, exposed to weather or corrosive atmosphere, shall be protected by a coating of asphaltic material.

Sec. 2638.

a) **Post-tensioned construction** shall conform to all requirements outlined in Sections 2633, 2634 and 2635 of this Code.

b) Values of total elongation, corrected for anticipated friction loss and anchorage set, and corresponding jack pressures at various increments of prestress shall be supplied by the contractor. In case of a difference in excess of 5% between anticipated elongation and jack pressure, the cause of the discrepancy shall be investigated and brought to the attention of the Engineer.

c) Prestressing steel to be post-tensioned shall be placed in openings or enclosures for which tubing, metallic casing or other materials may be used if approved by the Engineer. These enclosures shall be positioned accurately and tied solidly to prevent misplacement during pouring or vibrating of concrete.

d) Before tensioning, the tendons shall be moved in their ducts, or other methods employed to assure that the tendons are not bonded.

e) Parallel wire and strand assemblies shall be full length without splices, except that manufacturing splices in individual wires of a strand are permissible when made and spaced in accordance with strand specification U.B.C. No.
If couplers are used for extending alloy steel bar tendons, they shall be capable of developing the rated strength of the tendon when one coupled bar is pulled at an angle of 2 degrees with respect to the other. However no coupler shall be placed in areas of maximum bending moment or of moment reversal, unless special approval is obtained from the Engineer. Couplers shall be enclosed in a housing long enough to permit the necessary elongation.

f) Spacing between ducts shall generally be one and one-half times the aggregate size. However, if necessary, two ducts may be in contact horizontally, providing such grouping would not prevent proper placing of the concrete. Ducts may be in contact vertically if provisions are made so that a tendon will not break through into an adjacent duct when tensioned.

g) Spacing between ducts at the end block shall provide sufficient space so that anchorages can be located to comply with all provisions of this code.

h) When more than one tendon is required to provide the required prestressing force, a tensioning sequence shall be established which will ensure that eccentricities and unanticipated stresses are kept to a minimum during stressing.

i) The supplier of the anchorage shall submit detail drawings and shall state the amount of slip normally expected in seating the anchorage device.

j) Tendons may remain unbonded in their ducts after stressing, if fully protected against corrosion. Galvanized wires or strands may be considered protected when coated with grease or asphalt-impregnated material and enclosed in a sheath: If the steel is not galvanized, the coating shall protect the steel from corrosion during shipment, storage, construction and after the steel is in place. It shall permit movement of the steel during stressing with a minimum friction. Coating of non-galvanized steel shall be approved by the Engineer before installation.

k) Bonded tendons may be ungalvanized. Protection shall be obtained by filling the ducts with grout upon completion of the prestressing operation. Grout shall have the consistency of thick paint. It shall be pumped into the duct from one end, free passage at the other end or at an intermediate discharge point must be assured. For long beams devices for bleeding air shall preferably be installed at intermediate points. Grout shall be pumped with sufficient pressure to ensure that all voids are completely filled.

Sec. 2639. Design of composite structures

a) Physical properties of the composite section shall be computed, assuming complete interaction between component elements.

b) For structures composed of concretes of different qualities, the following considerations shall be observed:
   1) The area of one of the component elements shall be transformed in accordance with the ratio of the two moduli of elasticity.
   2) The allowable stresses shall be computed on the basis of the strength of the portion under consideration.

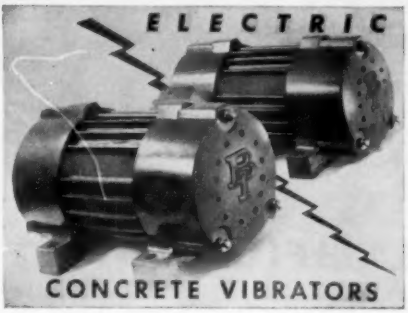
c) Ultimate strength of a composite section shall be computed in the same manner as ultimate strength of an integral member of the same shape.

Sec. 2640. Continuous prestressed concrete members.

a) Moments, shears and thrusts produced by external loads and prestressing force shall be determined by elastic analysis. Effects of axial deformation shall be investigated. Consideration of effects produced by the prestressing forces shall take into account the restraint of attached structural elements and supports.

b) Frictional losses in continuous members may be more significant than in simply supported members. They shall be given careful attention. When prestressing is to be applied in more than one stage, internal stresses shall be investigated at each stage.

c) Ultimate strength of a continuous member shall be evaluated at points of maximum moments as well as at intermediate points. At points where passage of live load causes moments of opposite sign to that of dead load, consideration shall be given to load factor combinations in which the dead load factor may equal one.



From a lifetime of experience in placing and compacting by vibration, comes this line of finest electric vibrators.

The reason for their superior performance is two-fold: First, because of the new type of SKF bearings especially designed for this vibrator, and second, because BI Vibrators are fan-cooled.

FULLY GUARANTEED!

Our BI Vibrators are fully guaranteed for performance perfection. Any burned-out vibrator, serviced according to instructions, will be replaced with a new one without charge.

EXPERTLY SERVICED!

European trained specialized mechanic will service your BI Vibrators after every 200 working hours of continuous or occasional operation for 3 years.

TWO TYPES FURNISHED:

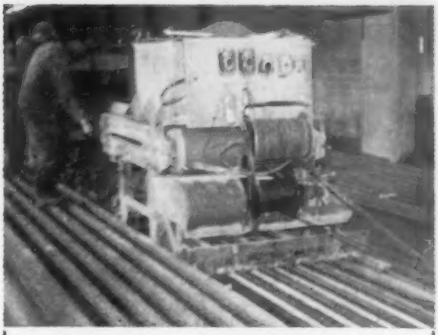
Our "Low Frequency" BI Vibrator Type "A" operates on your plant power line at 3400 RPM or frequency and at a variable, adjustable amplitude or impact ranging up to 10,000 pounds.

Our "High Frequency" BI Vibrator Type "B" operates through an electric current changer off your plant power line or a small generator in the field, on a frequency of 9,000 to 12,000 RPM and delivers a variable, adjustable amplitude up to 3,000 pounds.

BETON INSTITUTE Etablissement
Vaduz, Liechtenstein, Europe

Exclusive American Distributor:

GEORGE W. HOFFMANN
Post Office Box 272
Chatham, New Jersey, U.S.A.



The WEILER System AUTOMATIC EXTRUSION MACHINES

for the mass production of Prestressed Concrete Units

This WEILER prestressed concrete joist plant near Leeuwarden in Holland, makes about 20,000 feet of prestressed concrete joists a day on ten casting beds 360' long and 63" wide. The joists are extruded in rows of eight at a time.

The automatic WEILER extrusion machine is operated by only two men, and uses eight cubic yards of concrete per operating hour.

HOLLOW CONCRETE SLABS

Similar WEILER extrusion machines make hollow concrete slabs either one slab 4' wide or two slabs 2' wide at a time.

CONCRETE CHANNELS

In some European plants WEILER System extrusion machines make three or four at a time concrete channels, prestressed and finished for use without plaster coating.

Other such channels are extruded prestressed over lightweight concrete block, thus furnishing a lightweight insulating long span beam for school, hospital and home building construction in floors and ceilings.

For quotations, send complete specifications of the prestressed concrete building units you would like to make.

Our quotations will cover the complete plant for making the prestressing bed, the head and end wire abutments, the extrusion machine and accessories.

BETON INSTITUTE Etablissement
Vaduz, Liechtenstein, Europe

Exclusive American Distributor:

GEORGE W. HOFFMANN
Post Office Box 272
Chatham, New Jersey, U.S.A.

425 N. E. FIFTH STREET
BOCA RATON, FLORIDA

MEMBERSHIP QUALIFICATIONS

SCHEDULE OF DUES

Anyone engaged in the production of prestressed concrete, or in the production of materials or equipment allied to the prestressing industry; licensed architects and engineers; engineers and architects in training, and students enrolled in accredited schools of architecture and engineering, are invited to apply for membership in the Prestressed Concrete Institute. Membership classifications have been established as follows:

ACTIVE

Organizations and individuals actively engaged in the production of prestressed concrete products. — See schedule on application blank

ASSOCIATE

Organizations and individuals engaged in the production of materials and equipment allied to the prestressed concrete industry.
Annual Membership Dues — **$250.00**

PROFESSIONAL

Limited to registered architects and engineers
Annual Membership Dues — **$ 25.00**

AFFILIATE

Limited to non-professional personnel.
Annual Membership Dues — **$ 15.00**

JUNIOR

Limited to architects and engineers in training.
Annual Membership Dues — **$ 15.00**

STUDENT

Limited to students of accredited architectural and engineering schools. — **$ 10.00**
(For student members outside continental U. S., Canada included) — **$ 12.00**

Members receive the PCI JOURNAL, a quarterly technical publication, and PCItems, a monthly news bulletin, without additional cost, plus research bulletins and other information on prestressing as it becomes available.

425 N. E. Fifth Street, Boca Raton, Florida

APPLICATION FOR MEMBERSHIP IN THE PRESTRESSED CONCRETE INSTITUTE

(A non-profit corporation)

I am applying for membership in the classification checked:

☐ ACTIVE (producer).

Engaged in the following types of prestressing:

___Post-tensioning

___Pre-tensioning

___Combination of both

___On-the-job post-tensioning

___On-the-job pretensioning

___Precasting ___Design

ACTIVE MEMBERSHIP DUES (annually)

Based on Annual Prestress Business

$250 for	$ 100,000 or less		$650 for	$1,500,001 to	$2,000,000
350 for	100,001 to	500,000	750 for	2,000,001 to	2,500,000
450 for	500,001 to	1,000,000	850 for	2,500,001 to	3,000,000
550 for	1,000,001 to	1,500,000	950 for	3,000,001 to	3,500,000

$100 additional for each additional $500,000

☐ ASSOCIATE (related business). Membership Dues: $250.00 annually.

I am in the business of:_____

_____

☐ PROFESSIONAL (registered architect or engineer). Membership Dues: $25.00 annually.

I hold Certificate No._____of the State of_____

☐ AFFILIATE (non-professional personnel). Membership Dues: $15.00 annually.

☐ JUNIOR (limited to architects and engineers in training). Membership Dues $15.00 annually.

☐ STUDENT (limited to students of accredited schools of architecture or engineering).

Name of School_____

Membership Dues: $10 annually. Outside continental U.S.A. including Canada: $12 annually.

NAME OF COMPANY APPLYING FOR ACTIVE ASSOCIATE MEMBERSHIP_____

_____

ADDRESS_____

CITY_____ STATE_____

NAME OF REPRESENTATIVE IN INSTITUTE AFFAIRS_____

APPLICANTS FOR PROFESSIONAL, AFFILIATE, JUNIOR AND STUDENT MEMBERS PLEASE FILL IN BELOW:

NAME_____

YOUR MAILING ADDRESS_____

CITY_____ STATE_____

COMPANY AFFILIATION_____POSITION_____

DOES COMPANY MANUFACTURE PRESTRESSED CONCRETE?_____

Date_____ Signature_____

Make no payment until application is approved. Dues will be prorated from April 1 for all classifications.

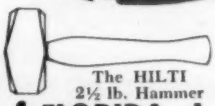
NOW ...the NEW FASTENING CONCEPT

the HILTI® DX-100 DRIVE TOOL

FOR EASY, SAFE FASTENING— EVEN OVERHEAD— INTO CONCRETE, ALL TYPES OF MASONRY

This entirely new hammer drive tool provides a booster-assist for fastening into extra dense materials; for lighter fastenings, no additional power is required. In either case, the blow of the hammer is all the force you need! Made of the finest tool steel, the DX-100 provides the means for driving a wide variety of pins and studs for your every fastening need.

HILTI "CONIC" PINS and STUDS



The HILTI 2½ lb. Hammer

HILTI of FLORIDA, Inc.

Rapid Fastening Systems Div.
2629 Central Ave.
St. Petersburg 13, Fla.

5th annual P.C.I. convention

NOVEMBER 1-7, 1959
DEAUVILLE HOTEL
MIAMI BEACH, FLORIDA

I **"P.C.I. AT WORK . . . Prestressing at Work"**

- ★ NEW TECHNIQUES
- ★ NEW DEVELOPMENTS
- ★ RESEARCH REPORTS
- ★ PANEL DISCUSSIONS
- ★ TECHNICAL PAPERS

II **DEAUVILLE HOTEL**
MIAMI BEACH, FLORIDA

- √ $6-11 per Person, per Day, Dble. Occ.
- √ On the Ocean
- √ $20,000,000 of Facilities for Your Enjoyment

III **POST-CONVENTION FIELD TRIP TO HAVANA, CUBA**

- ★ VISIT CASTING YARDS
- ★ INSPECT PRESTRESSED BUILDINGS AND BRIDGES
- ★ CITY SIGHTSEEING TRIPS
- ★ ALL-EXPENSE TRIP including air transportation and 3 days and 2 nights at Havana Hilton Hotel . . .

$86.50

IIII **FREE! ALL-EXPENSE TRIP TO HAVANA, CUBA**

Register for this convention before Sept. 15 and be eligible to win this wonderful prize!

IIIII **LADIES PROGRAM**

• Shopping Tours • Sightseeing Trips
• Luncheons • Fashion Shows
• Hospitality Corner

Register now for this important 5th annual convention of the Prestressed Concrete Institute. Registration fee is $25.00 . . . $15.00 for wives/husbands. Write, today to CONVENTION HEADQUARTERS, PRESTRESSED CONCRETE INSTITUTE, 3132 N. E. Ninth Street for official registration form. Do it now and be eligible for the terrific free trip to Havana, Cuba which will be awarded to some lucky individual who registers before September 15, 1959.

FOR CONSTRUCTION PERMANENCE...TEST!

KELLY BALL
Fast and simple method for determining the slump of freshly mixed concrete.
CT-386 PRICE $50.00*

SIEVE SHAKER
For fast, accurate sieving and grading of soils, aggregates, gravel and construction materials.
CL-392 PRICE $325.00*

AIR METER
Direct readings of entrained air in concrete can be made quickly and accurately.
CT-126 PRICE $194.50*

SOILTEST Incorporated
4711 WEST NORTH AVENUE • CHICAGO 39, ILLINOIS • SPAULDING 2-6400
*ALL PRICES F.O.B. CHICAGO. WRITE TODAY FOR COMPLETE CATALOG.

CURRENT PCI PUBLICATIONS

Specifications for pre-tensioned prestressed concrete (Tentative) 4 pages 35c per copy	SPC-100-57T
Standard prestressed concrete beams for Highway Bridge Spans 30 ft. to 100 ft. Prepared by Joint Committee AASHO and PCI. 1 sheet 30c per copy	STD-101-57
Specifications for post-tensioned prestressed concrete (Tentative) 4 pages 35c per copy	SPC-102-58T
PCI Standards for Prestressed Concrete Plants (Tentative) 12 pages in covers: $1.00 per copy (50c to PCI members)	SPC-103-58T
Underwriters' Laboratories, Inc. Report R-4123-1, May 12, 1958 Precast Floor or Roof Double Tee Slab. 24 pages in covers. $2.00 per copy ($1.00 to PCI members)	R-104-58
Typical Prestressed Concrete Products 20 pages in covers: 10c per copy	STD-105-58
PROCEEDINGS FOURTH ANNUAL CONVENTION PRESTRESSED CONCRETE INSTITUTE. 162 pages in covers: $5.00 per copy ($2.50 to PCI Members)	PRO-106-59
STANDARD PRESTRESSED CONCRETE BOX BEAMS FOR HIGHWAY BRIDGES SPANS TO 103 FT.—Prepared by Joint Committee, AASHO-PCI—1 page: 30c per copy.	STD-107-59
STANDARD PRESTRESSED CONCRETE SLABS FOR HIGHWAY BRIDGES SPANS TO 55 ft.—prepared by Joint Committee AASHO-PCI—1 page: 30c per copy.	STD-108-59

Orders for all PCI Publications should be addressed to

PUBLICATION OFFICE
PRESTRESSED CONCRETE INSTITUTE
3132 N. E. Ninth Street, Fort Lauderdale, Florida

Check or money order should accompany order and be made payable to:
PRESTRESSED CONCRETE INSTITUTE

P. C Items

Published monthly by

Prestressed Concrete Institute

This lively illustrated news bulletin presents news of Institute activities . . . views of members facilities . . . highlights outstanding prestressed concrete applications, and reports on latest developments in research, production of prestressed concrete.

Subscription $2.50 per year

PCItems
3132 N. E. Ninth Street
Fort Lauderdale, Florida

CONCRETE TESTERS

The World's Finest Low-Cost Precision Testers

For
CYLINDERS
CUBES
BLOCKS
BEAMS
PIPE

IF IT'S A CONCRETE TESTER YOU NEED-GET IN TOUCH WITH

FORNEY'S, Inc.
TESTER DIVISION
P. O. BOX 310 - NEW CASTLE, PA.

PROFESSIONAL
service **advertising**

FREDERICK C. LOWY
Consulting Engineer
Designs in Prestressed Concrete
Prestress Alternates for
Conventional Design
Design of Hydraulic Systems
259 W. 14th St. New York 11, N. Y.

LEAP ASSOCIATES
Consultants to the Prestressing Industry
H. H. Edwards, President
Lakeland, Florida
P. O. Box 1053 **Ph. MU 6-7143**

RITCHIE & CROCKER ENGINEERS, INC.
Structural Consultants
Fort Lauderdale **West Palm Beach**
Consultants in Prestressed Concrete
Reinforced Concrete — Steel Design
3132 N. E. Ninth St., Fort Lauderdale, Florida

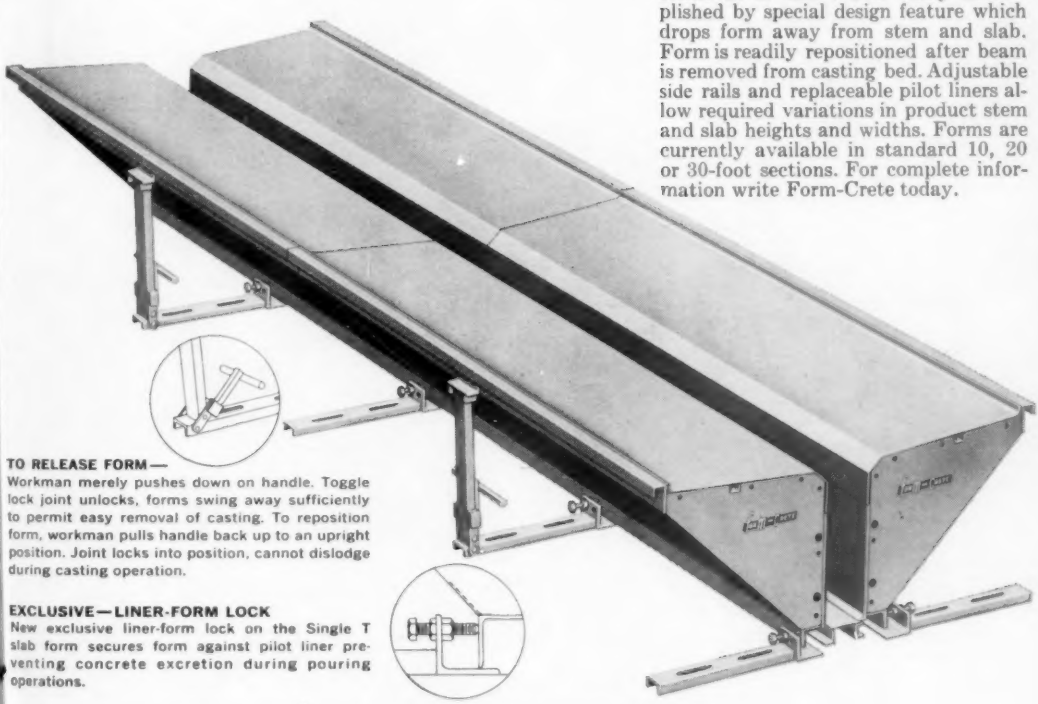
FREDERIC A. NASSAUX ASSOCIATES
Consulting Engineers
Room 312 Trust Bldg., Chambersburg, Pa.
317 Crain Hwy., Baltimore, Md.

FORM-CRETE LEADS THE WAY

NEW!

Form-Crete SINGLE T SLAB FORM

Here is the latest in form design from Form-Crete. Developed to conform to the latest industry specifications this new single-T form permits casting of a beam with either a six or eight-foot slab width. Removal of cast T's is easily accomplished by special design feature which drops form away from stem and slab. Form is readily repositioned after beam is removed from casting bed. Adjustable side rails and replaceable pilot liners allow required variations in product stem and slab heights and widths. Forms are currently available in standard 10, 20 or 30-foot sections. For complete information write Form-Crete today.



TO RELEASE FORM—
Workman merely pushes down on handle. Toggle lock joint unlocks, forms swing away sufficiently to permit easy removal of casting. To reposition form, workman pulls handle back up to an upright position. Joint locks into position, cannot dislodge during casting operation.

EXCLUSIVE—LINER-FORM LOCK
New exclusive liner-form lock on the Single T slab form secures form against pilot liner preventing concrete excretion during pouring operations.

TAKE ADVANTAGE OF FREE ENGINEERING HELP

Form-Crete engineers have long worked with designers and engineers in development of both standard and special forms for prestressing and precasting desired beams. These years of valuable experience are at your disposal without cost or obligation. Oftentimes an existing form can be modified. And if a custom form is required, you can be sure of prompt careful manufacture.

---

fmc FOOD MACHINERY AND CHEMICAL CORPORATION ®

Putting Ideas to Work

Form-Crete Department

General Sales Offices:
Lakeland, Florida — Riverside, California

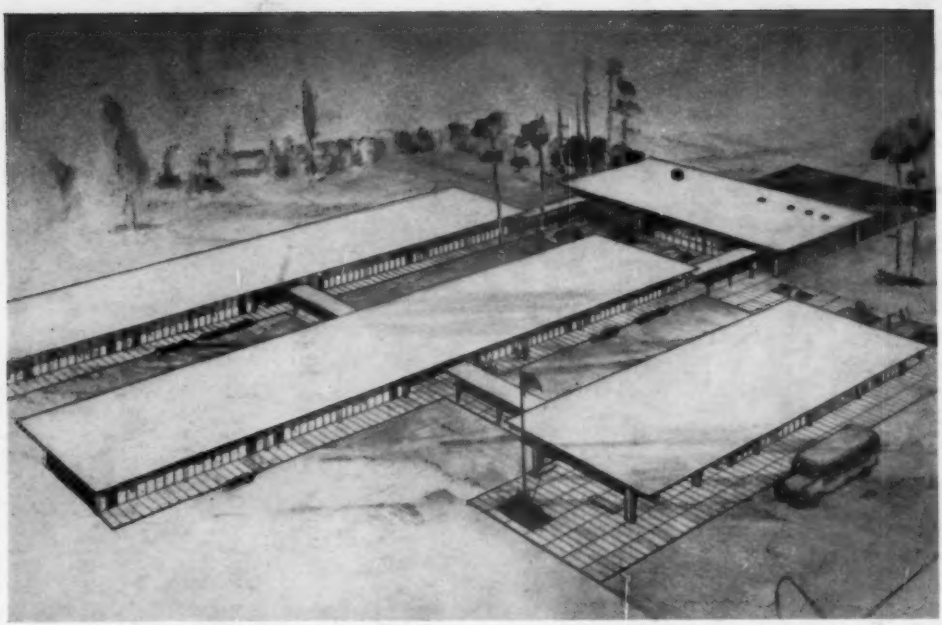
FOOD MACHINERY AND CHEMICAL CORPORATION — FORM-CRETE Department
LAKELAND, FLA. • RIVERSIDE, CALIF.

Please send us information on the new Form-Crete Single T form.

NAME _____

ADDRESS (RFD) _____

CITY _____ STATE _____



Normandy Elementary School, Jacksonville, Florida won an award at the Sarasota Regional A.I.A. Conference—*Architects:* George Ryad Fisher, A.I.A., and Harry E. Burns, Jr., A.I.A. *Contractors for 4 schools:* 1) McMaines Construction Co., Jacksonville; 2) East Coast Construction Co., Jacksonville; 3) Bay Construction Co., Norfolk, Va.; 4) Wm. S. Smith Construction Co., Jacksonville. *Prestressed Concrete Fabricator:* Capitol Concrete Co., Prestress Division, Jacksonville.

New Prestressed Concrete Florida Elementary School Wins A.I.A. Award

Low completed cost prompts School Board to build 3 additional schools of same design

This structure is approximately 35,360 sq ft in area and contains some 39,000 sq ft of prestressed Double T roof slabs. Over-all, the school represents about 300,000 cu ft at the contract price of $297,000; the cost per sq ft being $8.40. That's the cost benefit . . . there are others.

Further, the $8.40 cost per sq ft, together with the fireproof and maintenance-free character of the building, *so pleased the School Board that it has seen fit to build it, with slight modifications, on three additional sites.*

You see how the success of prestressed concrete repeats itself? This local acceptance of prestressed concrete is being duplicated throughout the nation. Not only in schools, but in office buildings, garages, warehouses, motels, bridges, piers, factories and transportation terminals.

Because Roebling pioneered in the development of prestressing elements and strand in the United States, we have much to tell you and a wealth of information and data to share with you. Your inquiry can be as general as "... all about prestressed concrete," or as specific as "... tensioning elements and casting bed data." In any case, we will be glad to furnish you with whatever you wish. Just address *any* inquiry to Construction Materials Division, John A. Roebling's Sons Corporation, Trenton 2, New Jersey.

Consult Roebling... *First* in the U. S. with prestressing and tensioning elements

ROEBLING

Branch Offices in Principal Cities
Subsidiary of The Colorado Fuel and Iron Corporation